INVESTIGATION OF BOVILLS LANDSLIP, NEAR DEVONPORT, TASMANIA

by

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This thesis contains no material which has been accepted for the award of a degree or diploma in any University and, to the best of my knowledge and belief, contains no copy or paraphrase of material previously published or written by another person, except where due reference is made in the text.

Alan Moon

Alan T. Moon

February, 1984

Aerial view from the north of the coastal scarp east of Devonport. Bovills Slip is in the centre of the photograph.

Aerial view of Bovills Slip. The edge of the slip is indicated by arrows.

PREFACE

The scientific study of earth slopes has applications ranging from problems in pure geomorphology to the prediction of slope stability for civil engineering purposes and the design of remedial measures where a landslip has destroyed or is threatening property, communications, or the lives of people.

Skempton and Hutchinson (1969) point out that in the study of natural slopes a proper understanding is required of four interrelated groups of topics:

1. recognition and classification of various types of mass-movements that can occur on slopes; their characteristic morphological features; their geological setting; their rates of displacement and the causes of failure;

2. classification and precise description of the materials involved in mass-movements, and the quantitative measurement of the relevant properties of these materials;

3. analytical methods of calculating the stability of a slope;

4. correlation between field observations and the results of stability calculations based on laboratory measured soil properties.

The fourth topic represents the sum of the previous three and is vitally important. Confidence in analytical methods and laboratory determined strength parameters can only be gained by careful back analysis of actual landslips. In this respect the work carried out at Imperial College, London in the past thirty years by Skempton, Hutchinson, Chandler, and many others, has been outstanding. They emphasised the importance of understanding the geological setting and geomorphological history of the slopes studied and have consistently tried to relate laboratory results back to what actually happens in the field. In the study of the stability of natural slopes they have effectively integrated the disciplines of geology, geomorphology, and engineering.

The purpose of this thesis is to present a similarly integrated case record of a Tasmanian landslip and thus contribute to the fourth topic listed above. There has been a concentrated effort on shear strength testing because effective strength parameters of Tasmanian soils have not previously been investigated in any detail.

The most interesting new aspect of the work was the recognition of different residual shearing mechanisms which enabled the relationship between shear strength parameters and plasticity index to be understood. The effective shear strength parameters obtained and the implications of the relationship of these parameters with the plasticity index have been discussed in two publications (Moon, 1983; and Moon, in press) which are presented with this thesis.

The writer is employed by the Department of Mines, Tasmania, and a secondary objective of this study was to review the work of the Department in the field of landslip investigations. Thus, although this thesis is primarily a detailed investigation of one active landslip, reference is also made to previous work on landslips in Tasmania and possible future research.

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ABSTRACT

Bovills Slip occurs in weathered basalt colluvium at the base of a coastal scarp about 2 km east of Devonport on the north coast of Tasmania. The colluvium consists of red-brown fissured silty clay with rock fragments. Many landslips occur in colluvial soils on the coastal scarp and also in basalt-derived soils elsewhere. Thus a detailed investigation and stability analysis of Bovills Slip is relevant to the general slope failure problem in Tasmania.

Pore water pressures measured with open standpipe piezometers show a correlation with rainfall, with peak pressures occurring during wet winter months.

Effective shear strength parameters were determined by both multi-stage direct shear tests and consolidated undrained triaxial tests with pore pressure measurements. Different residual shearing mechanisms were recognised in the shear box tests. Significantly different values of residual strength were associated with these different mechanisms. The fully softened strength parameters appropriate for the analysis of first-time landslips were investigated by both triaxial and shear box tests. For the soil tested both the residual and fully softened effective friction angles showed a pattern of dependence on the plasticity.

Surface movements have been monitored by repeated surveys, and subsurface movements have been monitored by regularly checking piezometer tubes for deformation. After heavy rain, in August 1981, the landslip moved by 20 to 30 mm.

A two dimensional model of the August 1981 failure has been analysed by limit equilibrium methods. The factor of safety is most sensitive to variations in piezometric head and cohesion. Analysis has

been used to assess the relative change in factor of safety (stability) caused by changes in the slope and by remedial measures. The stability was reduced when the slope was undercut by roadworks in 1973, and the first movements caused a decrease in shear strength of the soil. Downslope movements have produced shape changes which have tended to increase the factor of safety. Toe drainage, toe surcharge, and re-grading have already resulted in increased stability. Subsurface drainage, although effective, would be relatively expensive. Lime stabilisation and tree planting were also considered. In the long term well established trees may increase the factor of safety by as much as 50%.

Possible future research on landslips in Tasmania is discussed in order to demonstrate how the results of this detailed investigation may be used as a starting point for regional studies.

CHAPTER ONE

INTRODUCTION

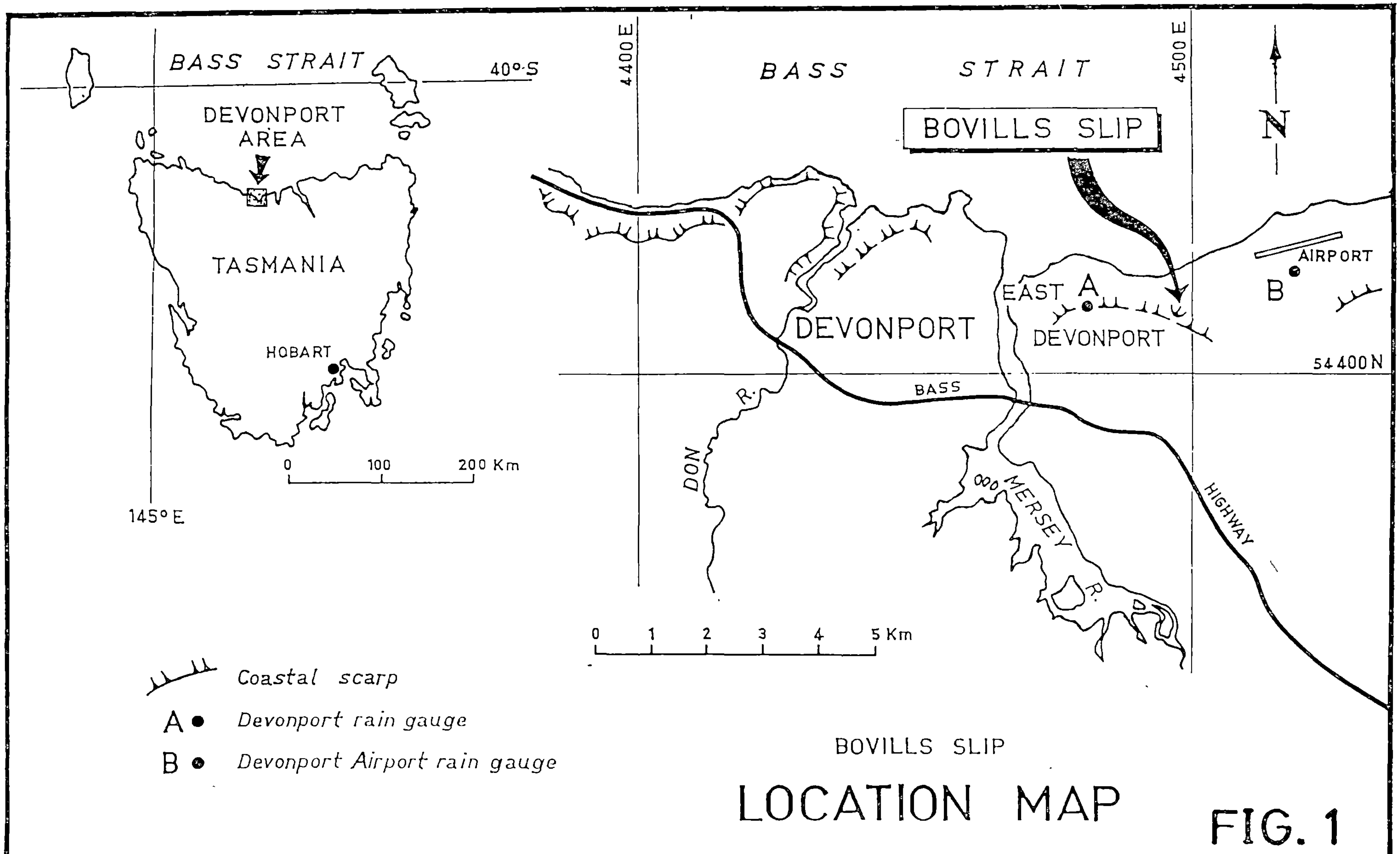
1.1 PREVIOUS WORK ON LANDSLIPS IN TASMANIA

Landslips commonly occur in stiff fissured clays in many areas of Northern Tasmania. In Launceston and the Tamar Valley the clays are lake sediments of Tertiary age. Along the north-west coast, a red-brown clay soil has developed on basalt of Tertiary age. Landslips have destroyed houses in several urban areas in Northern Tasmania. Landslips occur elsewhere in Tasmania on clay slopes, in colluvium, and in weathered rock.

The destruction of houses in urban areas has resulted in government legislation and the restriction of building in proclaimed landslip areas. Zone maps which advise users of relative landslip risk have also been produced. The investigation of proclaimed landslip areas and the risk zone mapping has been carried out by geologists from the Department of Mines, Tasmania (Stevenson and Sloane, 1980). Knights and Matthews (1976) described five landslips in the Tamar Valley and departmental geologists have investigated many individual landslips. The investigation of the St Leonards landslip near Launceston (Knights and Matthews, 1977) is the most detailed but many others have been recorded in Technical Reports and Unpublished Reports of the Department of Mines, Tasmania.

1.2 CHOICE OF LANDSLIP

The landslip chosen for detailed study occurs in colluvial soil developed on weathered basalt about 2 km east of Devonport on the north-west coast of Tasmania (Figure 1). The landslip has been named Bovills Slip after Mr W.Y. Bovill, the owner of the land on which it occurs.



BOVILLS SLIP

LOCATION MAP

FIG. 1

It was decided to study a landslip in basalt soil as landslips are common in this material and most previous studies have been on landslips in sedimentary clays in the Tamar Valley. In order to ensure that back analysis could be carried out it was necessary to choose an active landslip with a history of recent movement. It was hoped that back analysis would enable laboratory determined strength parameters to be compared with actual field strength at the time of failure. For this reason a landslip was chosen which appeared to involve only one type of material. The small size of Bovills Slip (about 3000 m^2) was also considered an advantage as it allowed a relatively intensive site investigation and monitoring programme to be carried out.

It was considered that successful back analysis was more likely to be achieved by a concentrated effort on one small landslip than by attempting to study a large complex landslip or many landslips over a wide area. If a small landslip could be understood, confidence could be gained in investigation techniques and the use of strength parameters which can then be applied to other landslips. Thus the successful unravelling of one case record can be considered the starting point for a regional understanding of landslips.

Recent movements of Bovills Slip began after roadworks at the base of the slope in 1973, and slip movements have been recorded in most subsequent years. Since remedial measures were carried out in 1977 and 1978 movements have been small. This study started in 1980 and the fact that Bovills Slip, while still active, did not urgently require further repair, was considered an advantage as it ensured that several years of uninterrupted monitoring could be achieved. The remedial measures in the past could also be subject to analysis and compared in their effect to any future measures which might be considered necessary.

1.3 LAYOUT OF THESIS

This thesis presents the results of a detailed investigation of Bovills Slip. The research project has involved field investigations of the geology, pore water pressures, rainfall, and slope movement. Laboratory investigations have included shear strength, grading, X-ray diffraction, density, and index property tests.

The main body of this thesis is in three parts. The first part (Chapters 2 to 6) presents and discusses the results of the investigations under the following general headings:

SHAPE OF SLIP	-	geological setting and geomorphological history, site geology.
WATER IN THE SLIP	-	pore water pressure and rainfall.
STRENGTH OF SLIP MATERIALS	-	shear strength parameters.
MOVEMENT OF SLIP	-	recent landslip movements.

The second part of the thesis (Chapter 7) presents the results of stability analyses, including sensitivity analyses, and consideration of the effects of slope modifications and remedial measures. The final part of the thesis (Chapter 8) summarises the study and presents suggestions for future research.

The basic data and the descriptions of the test methods are included in the Appendices. References to all sections of the work are included in the final appendix of this thesis.

1.4 TERMINOLOGY

The term *landslip*, or sometimes just *slip*, is used here to describe the mass-movement of earth materials on slopes. Landslides, slumps, and slump-earthflows are other terms which have been used elsewhere to describe similar mass-movements (Skempton and Hutchinson, 1969; Varnes, 1978). The particular landslip investigated in this study is known as

Bovills Slip. Different parts of the landslip have moved at different times and the terms *West Slip* and *East Slip* have been used to describe different parts of Bovills Slip (Figure 4).

The term *soil* is used in the engineering sense rather than the pedological. Thus all material that can be readily excavated with a pick or shovel is described as soil.

The terminology associated with the soil mechanics testing will be familiar to engineers but not necessarily to geologists and geomorphologists. The references will explain some of the terms, and important concepts have been explained where appropriate in the text.

At the base and sides of the landslip there is a *failure zone.* Some soil in the failure zone develops continuous *shear surfaces* or *slip planes* while other soil does not. The distinction between failure zones containing slip planes and failure zones which do not contain slip planes is important and the reader should be careful to recognise the different terms.

1.5 ADDITIONAL PUBLICATIONS

Two papers by the writer, which present some of the results of this research project, are included in Appendix H. The first paper, entitled 'Residual Shearing Mechanisms in Natural Soils' was published in the Special Edition of *Australian Geomechanics News,* pages 68-70, prepared for the International Society of Rock Mechanics Congress in Melbourne in 1983. The second paper is entitled 'Effective Shear Strength Parameters for Stiff Fissured Clays'. This paper will be presented at the *Fourth ANZ Conference on Geomechanics* in Perth in 1984 and will be published in the conference volume.

1.6 ACKNOWLEDGEMENTS

Many people have assisted the writer during the course of the research described in this thesis. To all of these people the writer extends his gratitude and appreciation. Special acknowledgement is recorded for the following people.

At the University of Tasmania, Eric Colhoun (Geography) and Brian Cousins (Civil Engineering) were my supervisors and I thank them for all their help, interest, and encouragement. Malcolm Gregory enthusiastically supported the project from the beginning and Ian Baldwin helped with some of the laboratory work.

In the Department of Mines, Richie Woolley helped with field work and laboratory testing and Richard Donaldson helped with the field monitoring programme. Loyd Matthews has more than twenty years' experience of working on landslips in Tasmania and his advice and comments on early drafts are gratefully acknowledged. Michael Dix prepared the frontispiece, reduced several of the figures, proof-read the final manuscript, and helped with the compilation of the thesis. The excellent job of typing was carried out by Claire Humphries.

The drafting of some of the figures was carried out by members of the Department of Mines Cartographic Section and the author acknowledges the assistance of John Ladaniwskyj, Peter Nankivell, Anthony Hollick, and Greg Dickens.

Ralph Rallings of the Department of Main Roads, and Tom Bowling of the Hydro-Electric Commission helped with laboratory work and in discussion on various aspects of the project. The Devonport City Council provided the services of a backhoe.

CHAPTER TWO

GEOLOGICAL SETTING AND GEOMORPHOLOGICAL HISTORY

2.1 INTRODUCTION

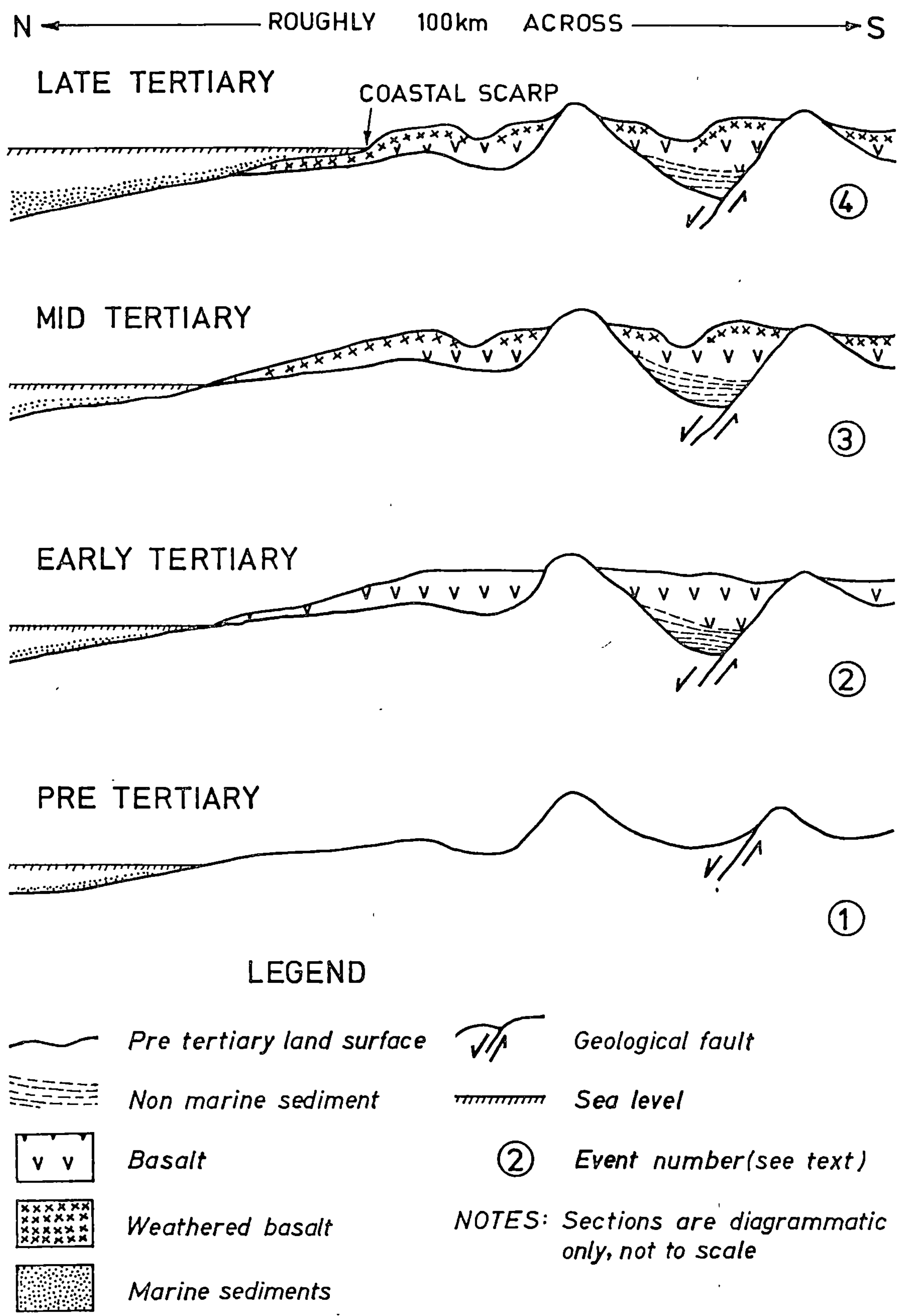
Bovills Slip occurs at the base of a coastal scarp formed in weathered Tertiary basalt. The main events in the geological evolution of the coastal scarp are summarised in Table 1, shown diagrammatically in Figures 2 and 3, and described in detail below.

TABLE 1

GEOLOGICAL EVOLUTION OF THE COASTAL SCARP

Period	Event	
	9	1973, road realignment undercuts slope.
	8	Holocene (post glacial), climate similar to present.
	7	Last Glacial, slope erosion, accumulation of colluvium.
QUATERNARY	6	Last Interglacial, sea level about 20 m above present level. Colluvium at the site of Bovills Slip removed by wave action in the intertidal zone.
	5	Earlier glaciations, slope erosion, accumulation of colluvium.
	4	Coastal scarp formed by marine erosion
TERTIARY	3	Weathering of basalts.
	2	Eruption of basalts.
	1	Pre-basalt land surface.

The exact timing of the events listed in Table 1 would be difficult to determine and is outside the scope of this project. However, for the purpose of this thesis it is assumed that the Tertiary period lasted from

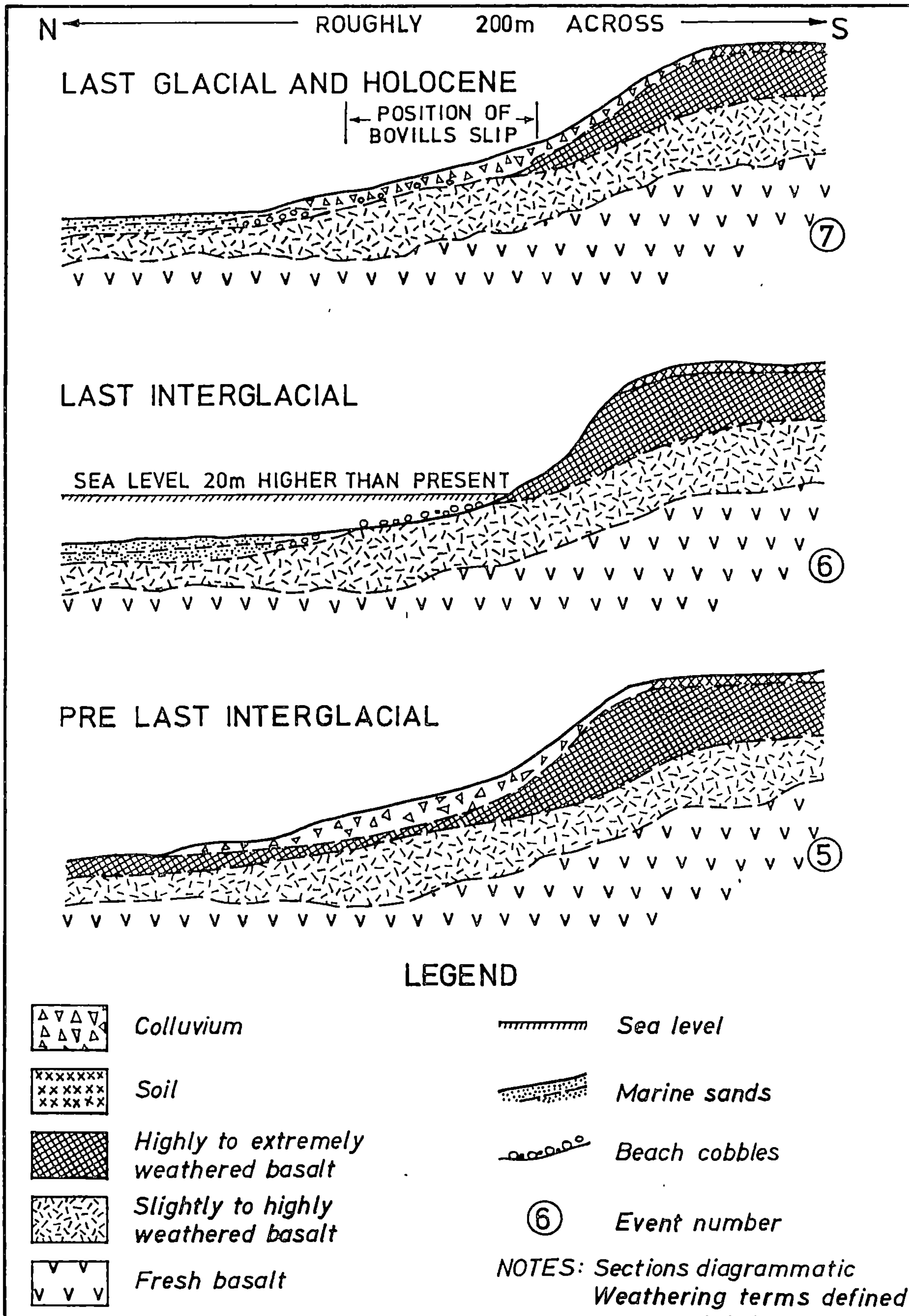


BOVILLS SLIP

TERTIARY HISTORY

SKETCH SECTIONS
NORTH-WEST COAST OF TASMANIA

FIG. 2



BOVILLS SLIP

QUATERNARY HISTORY

SKETCH SECTIONS OF COASTAL SCARP

FIG.3

70 to 2 million years before present (BP) and the Quaternary period lasted from 2 million years BP to the present day. The warmest part of the Last Interglacial was between 130,000 years and 120,000 years BP (Shackleton and Opdyke, 1973) and the Last Glaciation lasted from about 115,000 years to 10,000 years BP. The Holocene has been defined as the last 10,000 years as determined by radiocarbon dating (Bowen, 1978).

2.2 TERTIARY HISTORY

Prior to the Tertiary period the Devonport area was an eroded land surface underlain by sedimentary rocks of Permian age and by dolerite of Jurassic age (Figure 2, Event 1).

During the Tertiary period there were several phases of volcanic activity during which olivine basalts were extruded onto the land surface. Early flows tended to be restricted to the valleys while later flows were more extensive and submerged the lower interfluves (Figure 2, Event 2). During this period faulting produced basins. Lake and terrestrial sediments were deposited in these basins and in lava blocked valleys. Details of the geological history are given by Burns (1963 and 1964) and Cromer (1975 and 1980).

Throughout the Tertiary period weathering and erosion modified the landscape (Figure 2, Event 3). New valleys were formed and weathering altered the basalt lavas to depths of 30 m. The characteristic red-brown soils which overlie the basalts of Northern Tasmania were formed at this time. They are variously referred to as Krasnozems (Stace *et al.*, 1968) or as structured red earths with rough ped fabric (Northcote *et al.*, 1975).

The coastal scarp is a prominent feature on the north-west coast of Tasmania (Figure 1). It appears to have been formed by marine action during a period or periods when the sea level was higher than at present.

The age of the scarp is not known but a long period would be required for its formation. It is shown as Late Tertiary in Figure 2 (Event 4) but marine erosion at this level probably continued into the Quaternary.

2.3 QUATERNARY HISTORY

During the Quaternary period there have been many periods of colder climate. These have led to repeated glaciations in temperate parts of the world (Goudie, 1977) and many oscillations of sea level (Shackleton and Opdyke, 1973). There is evidence of at least two Quaternary glaciations in Tasmania and there may well have been more (Colhoun, personal communication). During these glaciations the coastal scarp east of Devonport was probably an unglaciated area even though close to the valley ice tongues that came down from Tasmania's Central Plateau. Mean temperatures are likely to have been at least 6°C colder than at present (Colhoun, personal communication).

Changes in climate would have caused changes in vegetation. The forest vegetation characteristic of temperate climates would have given way to open grassland and sparse woodlands during the colder periods. Root binding of soils would have been less and stronger frost induced processes would have affected the surface under conditions of reduced temperature. Solifluction (the slow downhill movement of soil associated with seasonally frozen ground) is likely to have affected the coastal scarp during the colder periods. Solifluction is thought to be caused by the high pore water pressures which develop when frozen soils thaw quicker than they can drain (Hutchinson, 1974). A grassed slope is also more vulnerable to slope wash erosion during periods of intense rain than a slope with a forest cover. Landslips and mudflows are other slope erosion processes which may have been more active during the colder periods. Although there is no direct evidence for Tasmanian slopes Grove (1972) presents historical records which show how the incidences of landslips and other slope erosion processes increased in Western Norway during the Little Ice Age between 1650 and 1760.

Colhoun (1976), in a description of Last Glacial Stage slope deposits, refers to soil inversion. He explains how an old soil profile can be inverted during slope erosion. Initially, the soil is stripped and moved downslope. This may expose weathered rock to frost action and subsequent transport by solifluction processes. Thus rock fragments may end up overlying transported and disturbed old soils. Concentration of rock fragments in the top 1.5 m of colluvium may be regarded as evidence of soil inversion at Bovills Slip. Dylik (1960) describes rhythmically stratified slope deposits which involved repeated inversions of the soil profile.

The coastal scarp prior to the Last Interglacial probably resembled the section shown in Figure 3, Event 5. Slope erosion processes had probably reduced the slope of the coastal scarp and had produced an accumulation of slope deposits or colluvium at the base of the scarp. Most of the colluvium is likely to have been deposited during the earlier periods of cold climate associated with glaciations in the mountains.

The warmest part of the Last Interglacial was between 130,000 years and 120,000 years BP (Shackleton and Opdyke, 1973) and there is evidence from several parts of the world that the sea level was higher than at present (Chappell, 1974; Fairbanks and Matthews, 1978). In Victoria the sea level was about 7 m above the present level (Gill, 1977) while in the Devonport area the sea level was about 20 m higher (Van der Geer, Colhoun and Bowden, 1979). Van der Geer *et al.* refer to these differences in Last Interglacial sea level highs in south-eastern Australia and suggest differential tectonic instability, and perhaps hydro-isostatic responses, may have affected Tasmania during Late Quaternary times.

The likely effect of the higher sea level on the coastal scarp is shown in Figure 3, Event 6. In the intertidal zone the colluvium and weaker weathered basalt were probably removed by wave action. Some beach cobbles were deposited at the base of the scarp (Section 3.3). The scarp was probably undercut and steepened and marine mud and sand were laid down on the floor of the bay.

During the Last Glacial Stage the sea level dropped to at least 100 m below the present level causing Bass Strait to be drained. Slope erosion processes would have been active during the colder periods, resulting in a flatter slope and a new deposit of colluvium (Figure 3, Event 7).

During the Holocene the coastal scarp has probably been relatively stable although landslips may have occurred during slightly wetter periods. Clearing of *Eucalyptus* forest after European settlement in the second half of the nineteenth century would have reduced stability (Section 7.7.3). The final stage in the evolution of the coastal scarp at the site of Bovills Slip follows modification of the base of the slope when the road was realigned in 1973 (Chapter 6). Bovills Slip appears to be located entirely in the colluvium which accumulated during the Last Glacial Stage (Section 3.4).

CHAPTER THREE

SITE GEOLOGY

3.1 INTRODUCTION

The site geology has been determined by surface inspection, logging of test pits and auger holes, and by a seismic refraction survey. The location of the test pits and auger holes is shown on Figure 4 and detailed logs are given in Appendix A. Details of the seismic refraction survey are given in Appendix B.

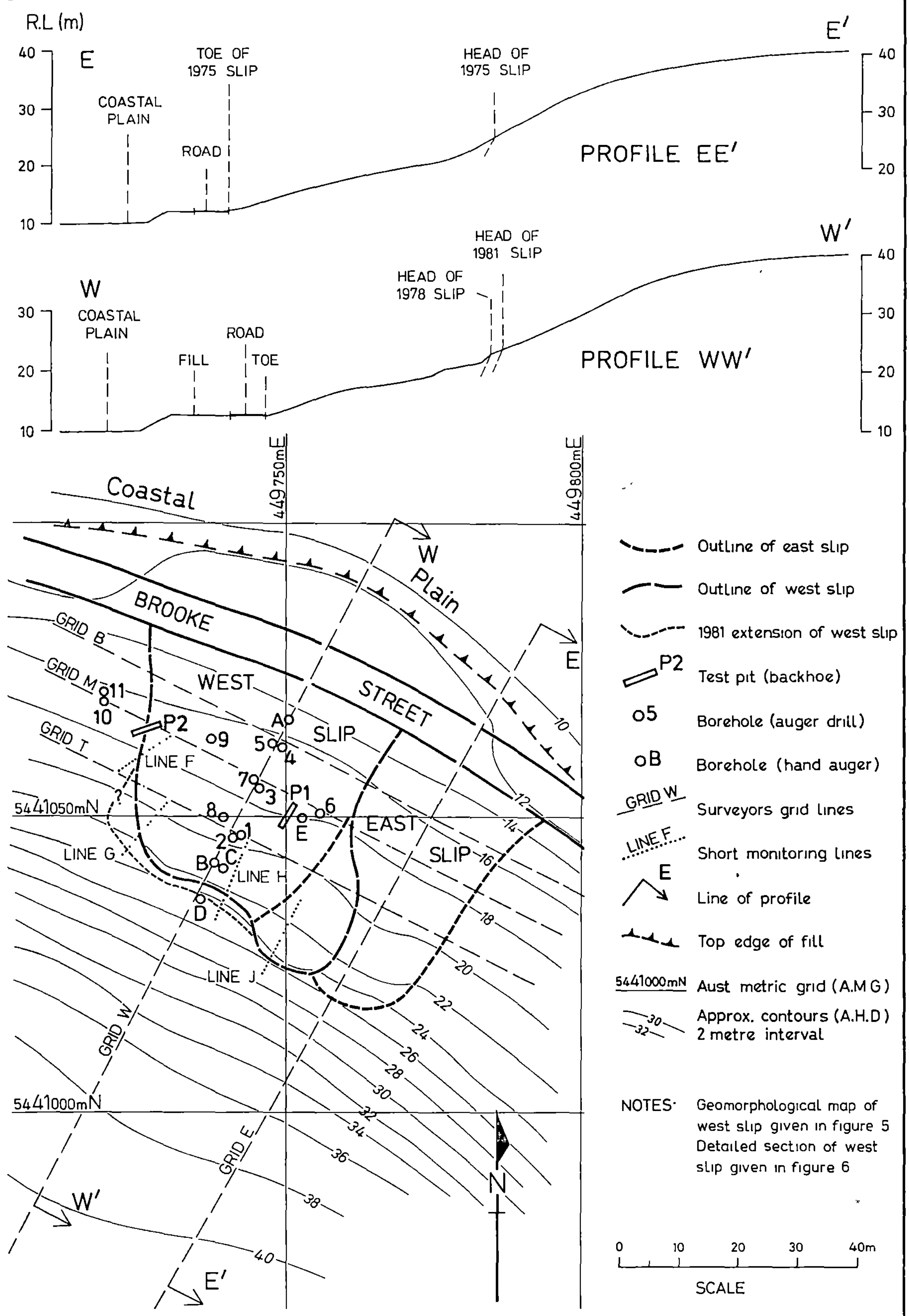
3.2 SURFACE CONDITIONS

Most of the surface of Bovills Slip is grassed. There are small bare patches of ground which expose red-brown, silty clay soil and sub-angular fragments of basalt (see Frontispiece). The steeper slope above the failed area is covered with eucalypts. The failed area of the slip has an uneven slope and is broken by steps and tension cracks. Surface details of the active slip are shown in Figure 5.

3.3 SUBSURFACE CONDITIONS

At the start of the project the East Slip appeared to be stable so work was concentrated on the still active West Slip (Figure 4). Figure 6 is a geological section of the West Slip. The colluvium is derived from weathered basalt. It consists of fissured, red-brown, silty clay with angular rock fragments. Locally there are variations in colour, plasticity, and in the proportions of rock fragments. Rock fragments make up less than 10% of the colluvium but are concentrated in the top 1.5 m. Several rounded quartzite pebbles were found between 2.4 m and 3 m in Borehole 5. These may have been derived from beach deposits formed along a shoreline suggested to be of Last Interglacial age.

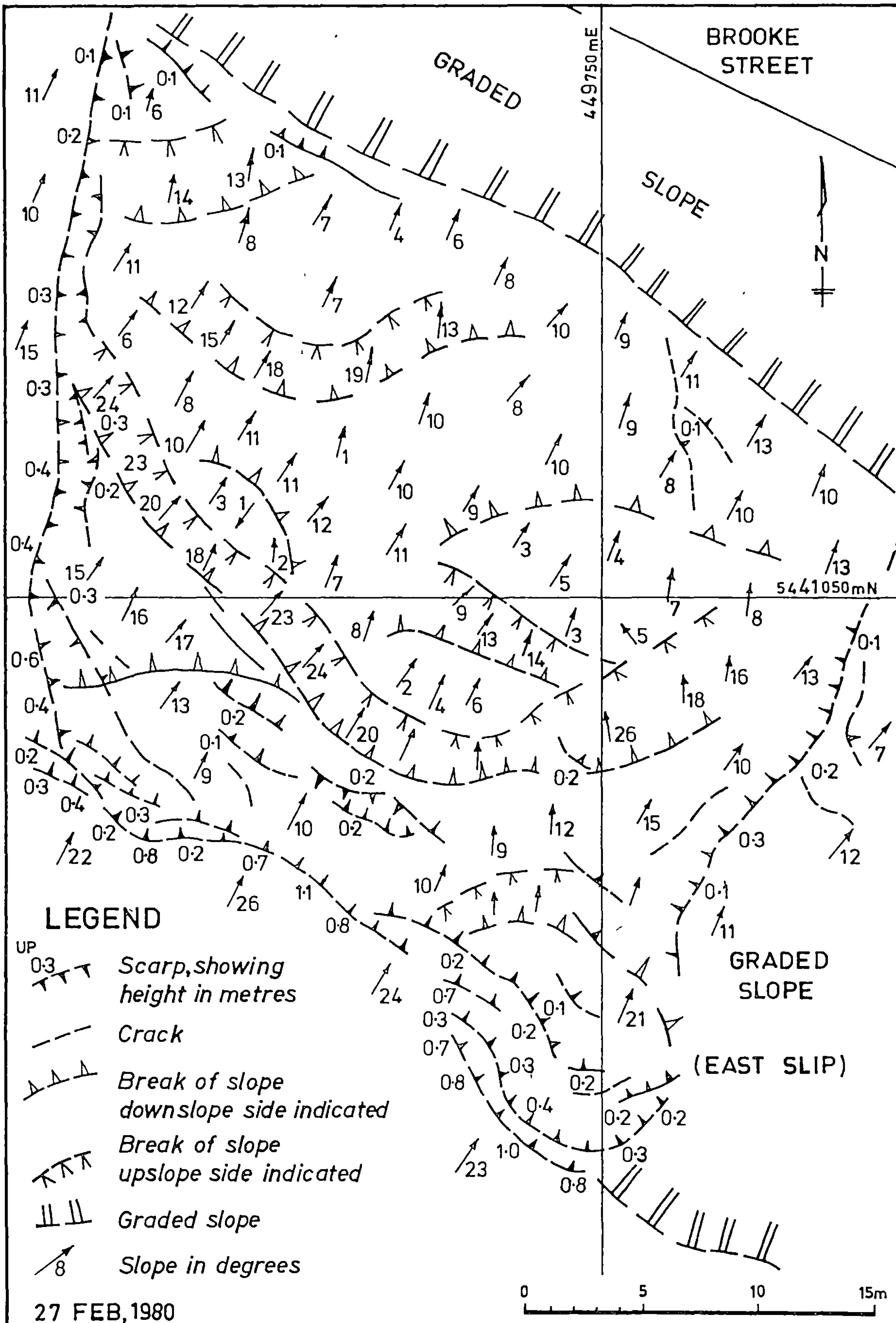
The profile below the colluvium is based on the interpretation of the seismic refraction survey (Appendix B). Most boreholes reached



BOVILLS SLIP

SITE MAP AND PROFILES

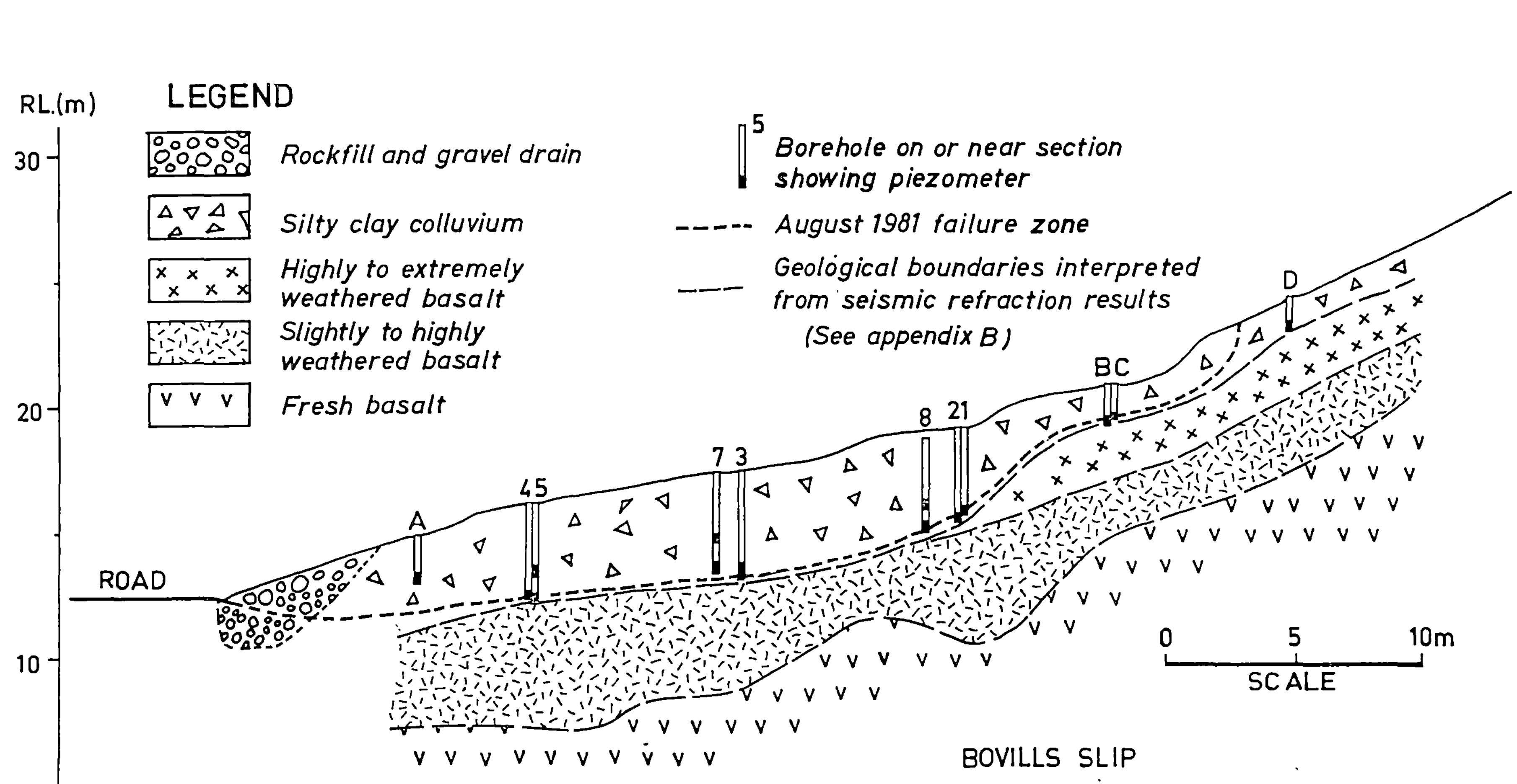
FIG. 4



BOVILLS SLIP

GEOMORPHOLOGICAL MAP OF WEST SLIP

FIG. 5



NOTE: Weathering terms are defined in Moon (1980)

GEOLOGICAL SECTION OF WEST SLIP

FIG. 6

the base of the colluvium but failed to penetrate the weathered basalt below. Extremely weathered basalt was found at the base of boreholes B, C and D. The type of profile indicated in Figure 6 has been picked up in water bores in the area. These bores indicate that basalt continues to below present sea level.

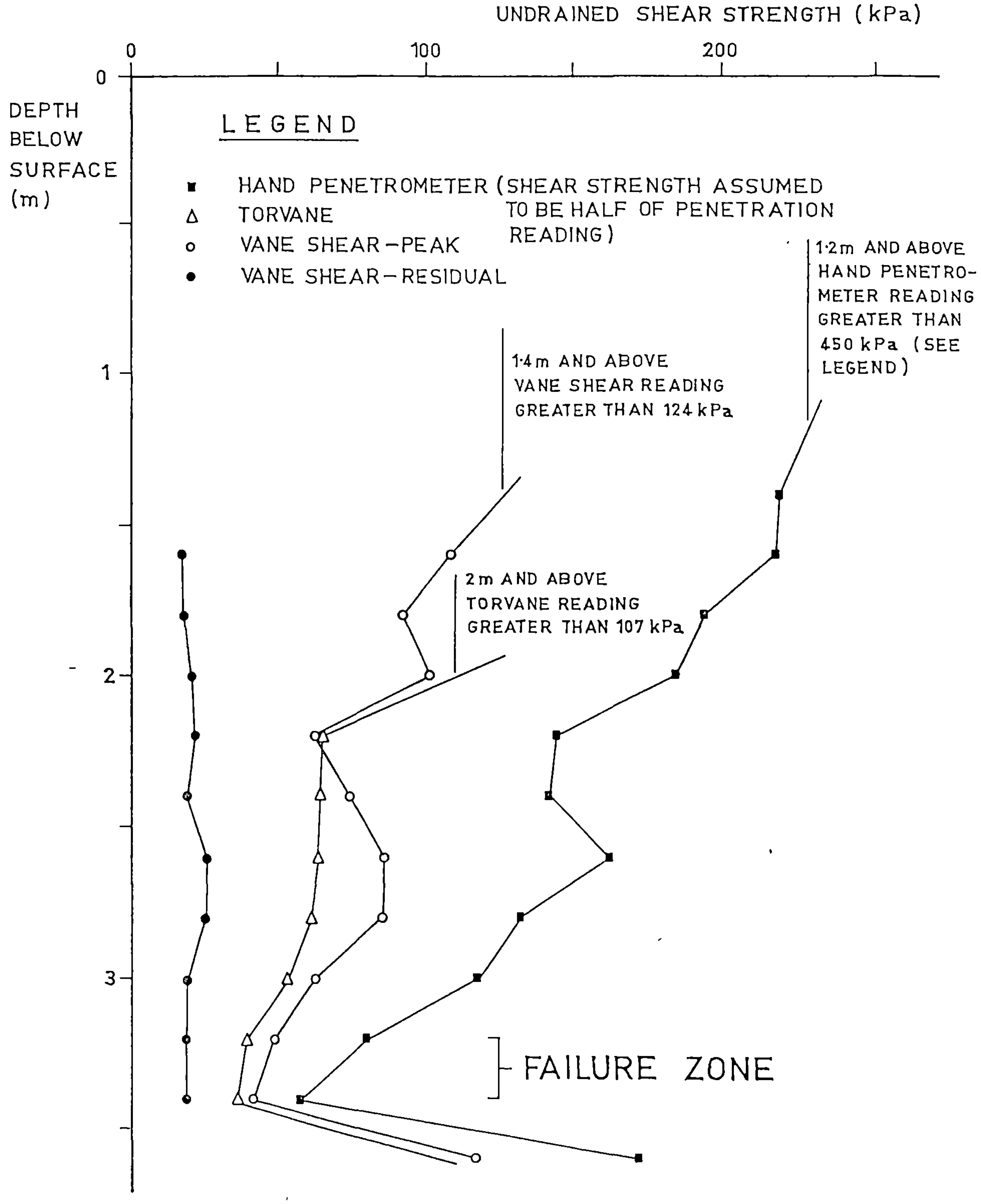
3.4 THE SHAPE OF THE LANDSLIP

The surface boundaries of Bovills Slip can be seen clearly (Figure 5) but the subsurface shape of the slip was more difficult to determine. Test pit 1 intersected the failure zone at the base of the slip. There was a small inflow of water and fissure surfaces were smooth but no continuous failure surfaces were seen. Test pit 2 straddled the edge of the slip. The edge was obvious at the surface but the failure zone could not be traced to depth in the side of the pit. The absence of continuous shear surfaces or slip planes and its implication is discussed in Chapter 5.

A second method of detecting the base of the slip was to assume that it coincided with softened zones. The colluvial soil at Bovills Slip has been overconsolidated by dessication. Thus the undrained shear strength is higher and the moisture content is lower than they would be for a soil normally consolidated under the present overburden pressure. If overconsolidated soil has failed the undrained shear strength in the failure zone should be lower than elsewhere in the soil (Chandler, 1974, and Hutchinson, 1983). Figure 7 shows that this method worked well. Undrained shear strength profiles (measured with a hand penetrometer, vane shear, and torvane), all picked up a softened zone which is assumed to coincide with the base of the slip. A softened zone was also observed in an undisturbed sample from Borehole 8. This zone coincided exactly with a zone of movement picked up later by monitoring.

The best way of picking up the subsurface shape of an active slip is by monitoring movement. This was successfully carried out using the PVC piezometer tubes (Appendix G).

The results of the geological investigation and the monitoring indicate that the landslip is located entirely within the colluvium and does not penetrate the weathered basalt (Figure 6).



BOVILLS SLIP

TEST PIT 1 EXPLORATION

UNDRAINED SHEAR STRENGTH PROFILES

FIG. 7

CHAPTER FOUR

PORE WATER PRESSURE AND RAINFALL

4.1 INTRODUCTION

Analysis of the long term stability of natural slopes should be carried out in terms of *effective stress* rather than *total stress* (Skempton and Hutchinson, 1969). For the reader unfamiliar with soil mechanics the fundamentally important concept of effective stress requires some explanation. The relationship between total stress, effective stress, and pore water pressure within an element of saturated soil is given by:

$$\sigma' = \sigma - u_w$$

where σ' is the effective stress

σ is the total stress

and u_w is the pore water pressure

The frictional strength which can be mobilised along the base of a landslip is proportional to the stress acting normal to the failure zone (normal stress). In the case of *total stress analysis* the normal stress is calculated from the total weight of soil above the failure zone. In the case of *effective stress analysis* the normal stress resulting from the weight of the soil is reduced by the uplift caused by the pore water pressure.

The uplift caused by the pore water pressure significantly reduces the available frictional strength. In conditions of horizontal or near horizontal flow the pore water pressure at any point is given by the piezometric head (or the depth below the piezometric surface) multiplied by the unit weight of water. If the unit weight of water is about a half of the unit weight of soil and the piezometric surface corresponds to the ground surface then the uplift pressure will be a half of the total stress and the available frictional strength will be halved. The important effect that changes in pore water pressure given by changes in piezometric head can have on the factor of safety against failure of Bovills Slip is discussed in Section 7.6 and shown in Figure 14.

The addition of water to soil which may not be fully saturated close to the ground surface will slightly increase the weight of the soil. The effect of this increase in weight at Bovills Slip is very small and has a negligible effect on the factor of safety (Section 7.6, Figure 13).

Pore water pressures vary with time and in a shallow landslip rainfall is the main cause of this variation. In this chapter the relationship between pore water pressure and rainfall is discussed.

4.2 MEASUREMENT OF PORE WATER PRESSURE, SOIL PERMEABILITY, AND RAINFALL

Pore water pressures have been measured with open standpipe piezometers. The design and location of the piezometers are discussed in Appendix C. In order to understand the relationship between the piezometer record and the actual pore water pressure in the soil at any particular time it is necessary to have some knowledge of the permeability of the soil. This was obtained by field permeability tests, the results of which are given in Appendix C. The time lag between a change of pore water pressure in the soil and the piezometer record of that change is also discussed in Appendix C.

Daily records of rainfall are available from two recording stations in the Devonport area (Figure 1) and a rain gauge was installed on the landslip for a short period. Rainfall records are discussed in Appendix C.

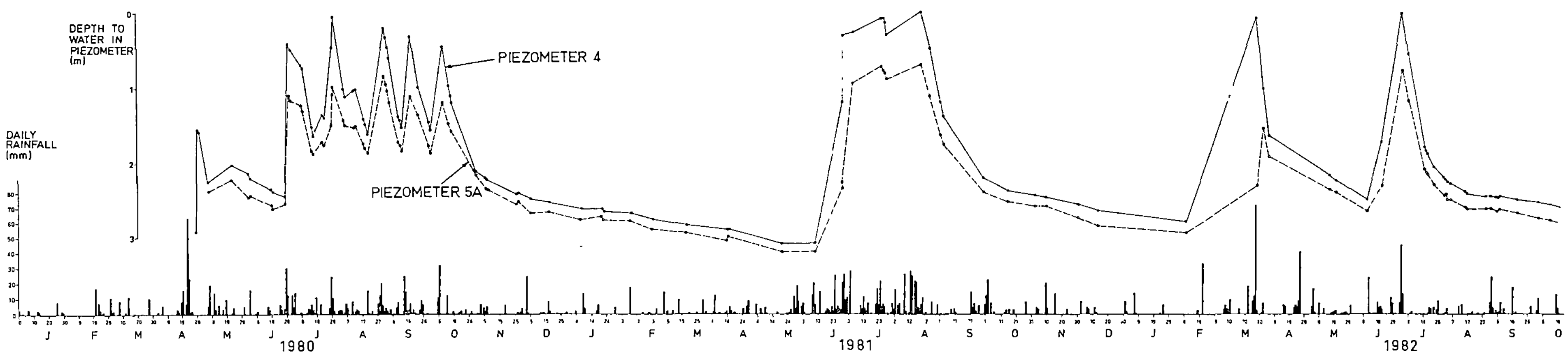
4.3 RELATIONSHIP BETWEEN PORE WATER PRESSURE AND RAINFALL

The relationship between pore water pressure and rainfall for two of the piezometers is shown in Figure 8. Similar records are available for all of the piezometers. There is clearly a correlation between pore water pressure and rainfall.

The water levels in the piezometers were recorded by an electrical probe. Intervals between readings varied from two hours to several weeks. If continuous records had been available there would have been more pore water pressure peaks on Figure 8. Because of the lack of continuous records an attempt has been made to develop a model to predict the variation of pore water pressure with rainfall. Given the initial pore water pressure and the rainfall the model predicts the new pore water pressure for a particular piezometer. Details of the model are given in Appendix C.

Although continuous records were not available during this study a simple method of measuring peak pressures was used. A thin metal strip painted with water colour was left in the piezometer. The water colour was removed when the water level rose, and the maximum level reached since the previous reading could be recorded. There are sufficient data on maximum water levels to suggest that pore water pressures at critical times may be estimated to within 2 or 3 kPa.

The effect of rainfall intensity has not been considered but with a shallow landslip and relatively permeable soils it is likely to be important. A 30 mm rainfall in one hour may have a different effect to 30 mm in 24 hours. Immediately following a short period of intense rain on 29th June 1981 some piezometers recorded rises in water level of over one metre in less than two hours.



BOVILLS SLIP

PIEZOMETERS AND RAINFALL

APRIL 1980 TO OCTOBER 1982

FIG.8

CHAPTER FIVE

SHEAR STRENGTH PARAMETERS

5.1 INTRODUCTION

Effective shear strength parameters are required for the analysis of the long term stability of natural slopes. These parameters are usually determined by either laboratory tests or the back analysis of existing failures. *Effective shear strength* parameters as opposed to *total* shear strength parameters can only be obtained if pore water pressures developed during the test or field failure are known.

Effective shear strength parameters were determined by multi-stage direct shear tests and consolidated, undrained triaxial tests with pore pressure measurements. Other laboratory work has included consolidation, classification and index, and density tests. Description of test procedures and full results of all the laboratory tests are given in the following Appendices:

Appendix D	-	Shear box tests
Appendix E	-	Triaxial tests
Appendix F	-	Other laboratory tests

In this chapter the definition of the parameters required for analysis is considered and the relationship between laboratory determined parameters and those applicable to the field is discussed. A relationship is demonstrated between the shear strength parameters and the plasticity index. Summaries of some of the test results are presented where necessary for discussion. The Appendices should be referred to for the full results and discussion of test details.

5.2 DESCRIPTION OF SOIL

All of the samples tested were obtained from test pits and boreholes within the landslip. Field observations and laboratory tests indicate that the slip occurs within one soil unit of constant clay mineralogy. The soil has a continuous variation in plasticity due to variations in clay content. The soil consists of red-brown silty clay with minor rock fragments. Soil properties are summarised in Table 2 and the detailed results of the classification tests are given in Appendix F.

TABLE 2

SOIL PROPERTIES

Liquid Limit:	46 to 124%
Plastic Limit:	28 to 44%
Plasticity Index:	17 to 84%
Clay Fraction:	30 to 65%
Activity:	0.53 to 1.28
Clay Mineralogy:	Montmorillonite and kaolinite

5.3 STRENGTH PARAMETERS REQUIRED

In the analysis of landslips in stiff fissured clays the soil strength available depends on whether there has been previous movement. If there has been no previous movement the soil has a higher strength than if past movements have occurred. In the case of Bovills Slip there is a history of landslip movement (Chapter 6) and present day movements are likely to be largely confined to pre-existing failure zones. Skempton (1964) demonstrated that *residual strength* parameters are appropriate for the analysis of such renewed movements.

If there has been no previous movement Skempton (1970) suggested that the field strength of a stiff fissured clay corresponded to the *fully softened* condition. This condition is reached when further deformation at constant stress fails to cause any further increase in water content. Skempton considered that the fully softened condition could be taken as a practical approximation of the critical state.

The peak strength of normally consolidated remoulded clay is also the theoretical minimum strength of a stiff fissured clay which has undergone complete softening.

In a review of the slope stability of cuttings in Brown London Clay, Skempton (1977) reported that the fully softened angle of friction is equivalent to the peak angle of friction determined by laboratory tests on undisturbed samples. However, values of cohesion determined in the laboratory generally over-estimate fully softened cohesion (C') Chandler and Skempton (1974) discussed the cohesion intercept obtained by back analysis, and argued that although the field cohesion at the time of first failure is small, it cannot be zero. They pointed out that the $C'=0$ assumption leads to the conclusion that the limiting slope of a cut would be, contrary to practical experience, independent of depth. They suggested C' values of between 1 and 2 kPa for London Clay and Upper Lias Clay. These values are similar to the residual cohesion determined by laboratory tests.

In light of the above discussion the effective shear strength parameters appropriate for the analysis of first time slips are referred to in this paper as the fully softened parameters. The fully softened angle of friction (ϕ') is assumed to be equal to the peak angle of friction determined by laboratory tests while the fully softened cohesion (C') is assumed to be equal to the cohesion obtained in residual strength tests.

5.4 RESIDUAL SHEAR STRENGTH

5.4.1 Test methods and procedures

Residual shear strengths of samples of silty clay colluvium were determined by multi-stage direct shear tests using a 60 mm square reversing shear box. A discussion of the choice of test type and a description of test apparatus and procedures is given in Appendix D.

5.4.2 Residual shearing mechanisms

Although all the tests were carried out on samples from one soil unit of constant clay mineralogy, the results of the tests led the writer to divide the samples into three groups. The majority of samples were placed in Groups 1 and 3 but there were two samples whose results suggested that an intermediate Group 2 existed.

Group 1 samples had a lower plasticity and a higher residual strength than samples from Group 3. Group 1 samples produced different load displacement curves from Group 3 samples with greater shear box displacement being required before flat curves were obtained (Appendix D). Group 3 samples developed polished and slickensided shear planes whereas Group 1 samples did not develop visible shear planes, even after 60 or 70 reversals. It was only after most of the shear box testing had been completed that the writer became aware of the work on residual shearing mechanisms by Lupini, Skinner and Vaughan (1981) which provided an explanation of the differences in behaviour of Groups 1 and 3.

Lupini *et al.* demonstrate how the behaviour of a soil in residual shear is controlled by the proportion of platy clay particles. Soils with a low proportion of clay fail by *turbulent* shear without the development of shear planes. Soils with a high proportion of clay fail by *sliding* shear and develop low shear strength surfaces of strongly oriented clay particles. Lupini *et al.* also describe a transitional mode which involves both turbulent and sliding shear. Lupini *et al.* worked with soil mixtures with artificially varied gradings. Electron micrographs and thin sections were used to examine the failure zones.

Comparing the results of the direct shear tests on the silty clay colluvium with the work of Lupini *et al.* it appears that Group 1 samples failed by turbulent shear, Group 3 by sliding shear, and Group 2 by a transitional mode.

Lupini *et al.* also reviewed published correlations between residual friction angles and index properties. They concluded that although such correlations cannot be general they may be useful in studying particular variable soil deposits.

5.4.3 Residual shear strength results

Residual strength results for fifteen different samples are summarised in Figure 9 and in Table 3. Detailed results for individual samples are given in Appendix D.

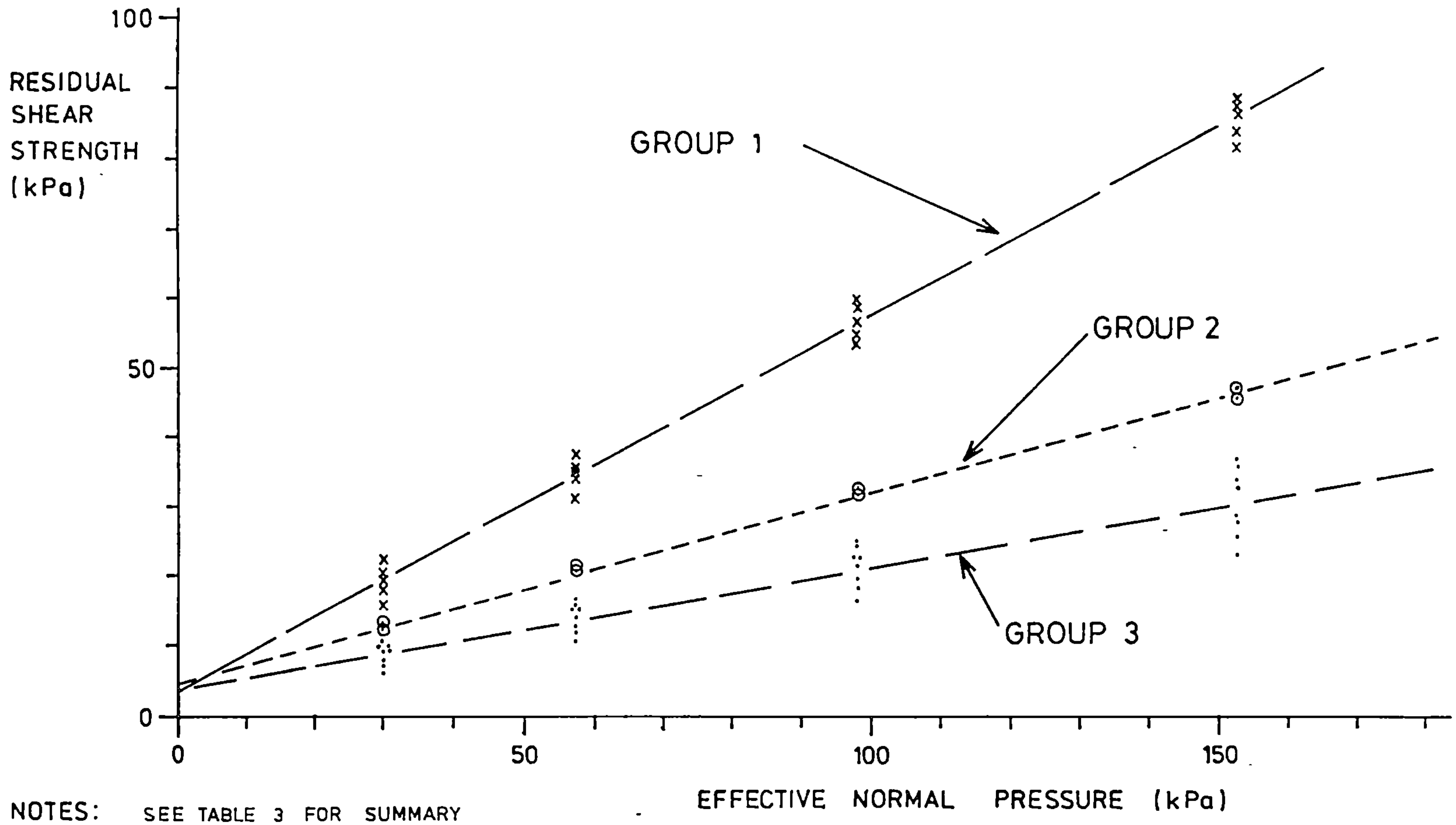
TABLE 3

SUMMARY OF RESIDUAL SHEAR STRENGTH RESULTS

Group number	Shearing mechanism	Number of tests	Residual cohesion C_r' (kPa)		Residual friction angle ϕ_r'		R^2 (%)
			mean	95% confidence limits	mean	95% confidence limits	
1	turbulent	5	3.6	1.1 to 6.1	28.3	27.1 to 29.4	100.00
2	transi-tional	2	4.9	3.3 to 6.5	15.2	14.3 to 16.1	99.93
3	sliding	8	3.7	1.3 to 6.0	10.0	8.6 to 11.3	99.94

NOTE: R^2 is a measure of the proportion of variation in the data that is explained by the assumption that the regression equation is linear.

The relationship obtained between the residual shear strength and the plasticity index (Figure 10) follows a similar pattern to that obtained by Lupini *et al.* (1981) for artificial soil mixtures. Up to a plasticity index of about 40% the samples failed by turbulent shear and shear planes did not develop even after many reversals. Above a plasticity index of 50 to 60% the samples failed by sliding shear and developed polished and slickensided shear planes. The two intermediate results may be regarded as representing the transitional mode.



NOTES: SEE TABLE 3 FOR SUMMARY OF GROUPS.

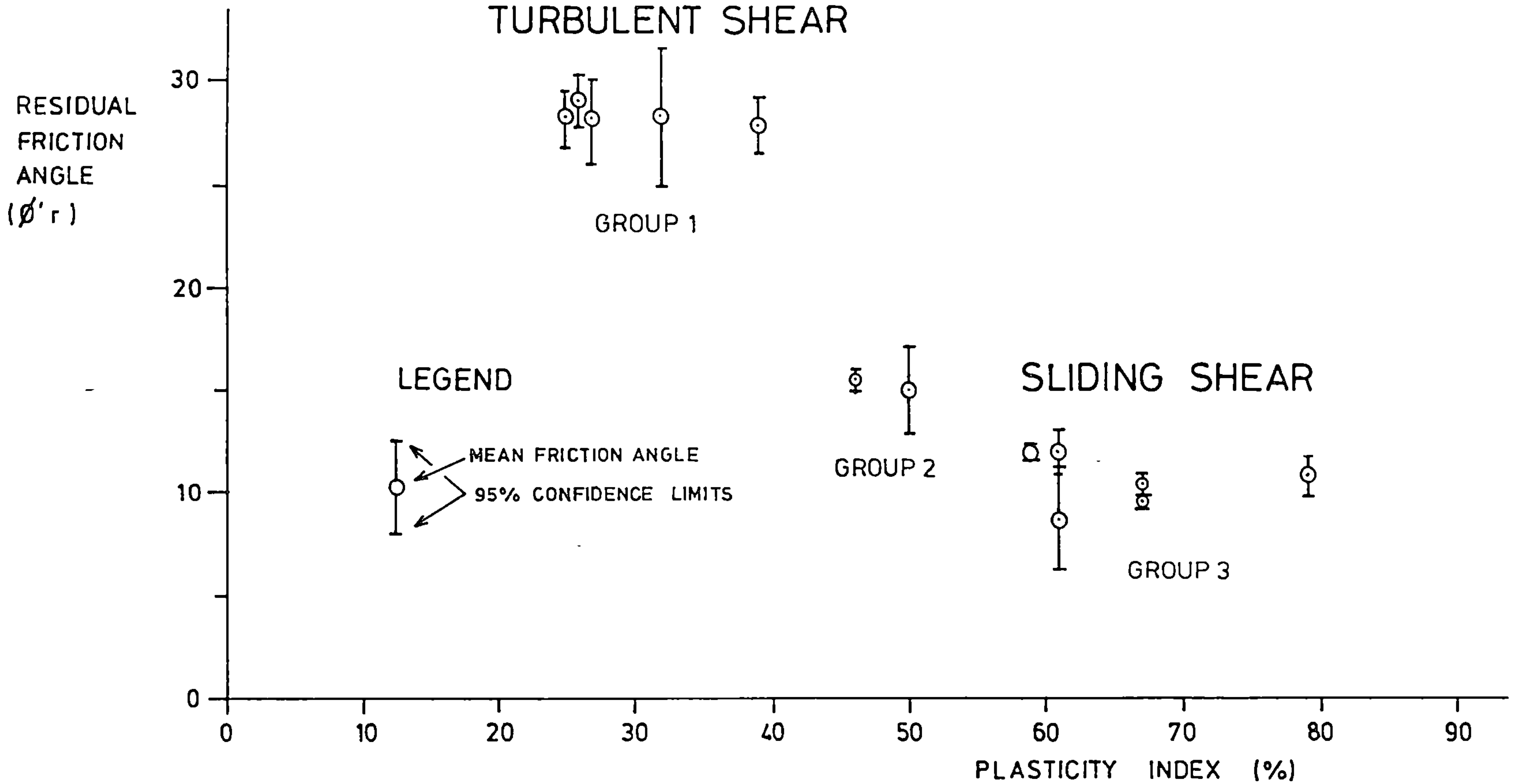
DETAILED RESULTS OF DIRECT SHEAR TESTS ARE GIVEN IN APPENDIX D

BOVILLS SLIP

DIRECT SHEAR TESTS

SUMMARY OF RESIDUAL SHEAR STRENGTH RESULTS

FIG. 9



BOVILLS SLIP

RESIDUAL STRENGTH AND PLASTICITY

FRICTION ANGLE V PLASTICITY INDEX

FIG. 10

In Bovills Slip most of the colluvium had a plasticity index in the lower part of the range (Section F.2, Appendix F). Thus it is likely that most of the failure zone will be located in colluvium which failed by turbulent shear. Continuous shear surfaces or slip planes do not develop during turbulent shear. This means that although there is a softened failure zone (Section 3.4) there are not likely to be continuous shear surfaces or slip planes under most of the slip despite the fact that there is a history of repeated movements over several years (Chapter 6).

5.5 FULLY SOFTENED SHEAR STRENGTH

5.5.1 Test methods

Fully softened shear strength parameters were investigated by consolidated undrained triaxial tests and by direct shear tests. As discussed earlier (Section 5.3) laboratory strength testing on undisturbed samples may be expected to provide an estimate of the fully softened angle of friction (ϕ') but will generally over-estimate the fully softened cohesion (C'). The five different methods used to determine ϕ' are shown in Table 4.

Tests on undisturbed samples were preferred to tests on remoulded samples because remoulding destroys any diagenetic bonds or preferred particle orientation which may occur in natural soils.

TABLE 4

METHODS USED TO DETERMINE FULLY SOFTENED STRENGTH

Apparatus	Sample Type	Failure Definition
Triaxial	undisturbed	maximum ratio of principal stresses
Triaxial	undisturbed	maximum difference of principal stresses
Shear box	undisturbed	peak strength
Shear box	undisturbed	post peak strength (at 7 mm displacement)
Shear box	remoulded	peak strength of normally consolidated sample

5.5.2 Fully softened shear strength results

The results of the investigation of fully softened strength parameters by triaxial and shear box testing are summarised in Table 5. Soils with a plasticity index of less than 40% had a higher strength than soils with a plasticity index of 50% or greater. Thus the results were divided into two groups and analysed separately. The fact that the different methods of estimating ϕ' gave similar results increases confidence in the parameters obtained.

Details of the triaxial test methods, procedures, and results are discussed in Appendix E and details of the peak, post peak, and remoulded shear box tests are given in Appendix D.

5.6 RELATIONSHIP BETWEEN SHEAR STRENGTH PARAMETERS AND PLASTICITY INDEX

The relationship between angle of friction (ϕ') and plasticity index (PI) for the soil tested is shown in Figure 11. The post peak results were obtained by analysing groups of samples with similar plasticity. Group A represents ϕ' obtained by linear regression analysis of test results obtained on eleven samples whose PI ranged from 25 to 33%.

TABLE 5

RESULTS OF TESTS USED TO INVESTIGATE FULLY SOFTENED STRENGTH

Test Method	Plasticity index less than 40%				Plasticity index 50% or greater			
STAGED TRIAXIAL	Cohesion in kPa	Friction angle	R^2 %	Number of samples	Cohesion in kPa	Friction angle	R^2 %	Number of samples
Maximum ratio of principal stresses	14.4	30.8	99.95	1	8.2	22.0	98.72 to 99.60	3
Maximum difference of principal stresses	20.0	28.4	99.89	1	9.4	20.5	97.53 to 99.93	3
SHEAR BOX								
Peak	6.5	30.6	99.26	12	15.7	22.9	95.06	9
Post peak	2.8	30.4	99.76	12	7.8	20.7	99.91	9
Remoulded	-	-	-	-	6.5	19.6	99.38	1

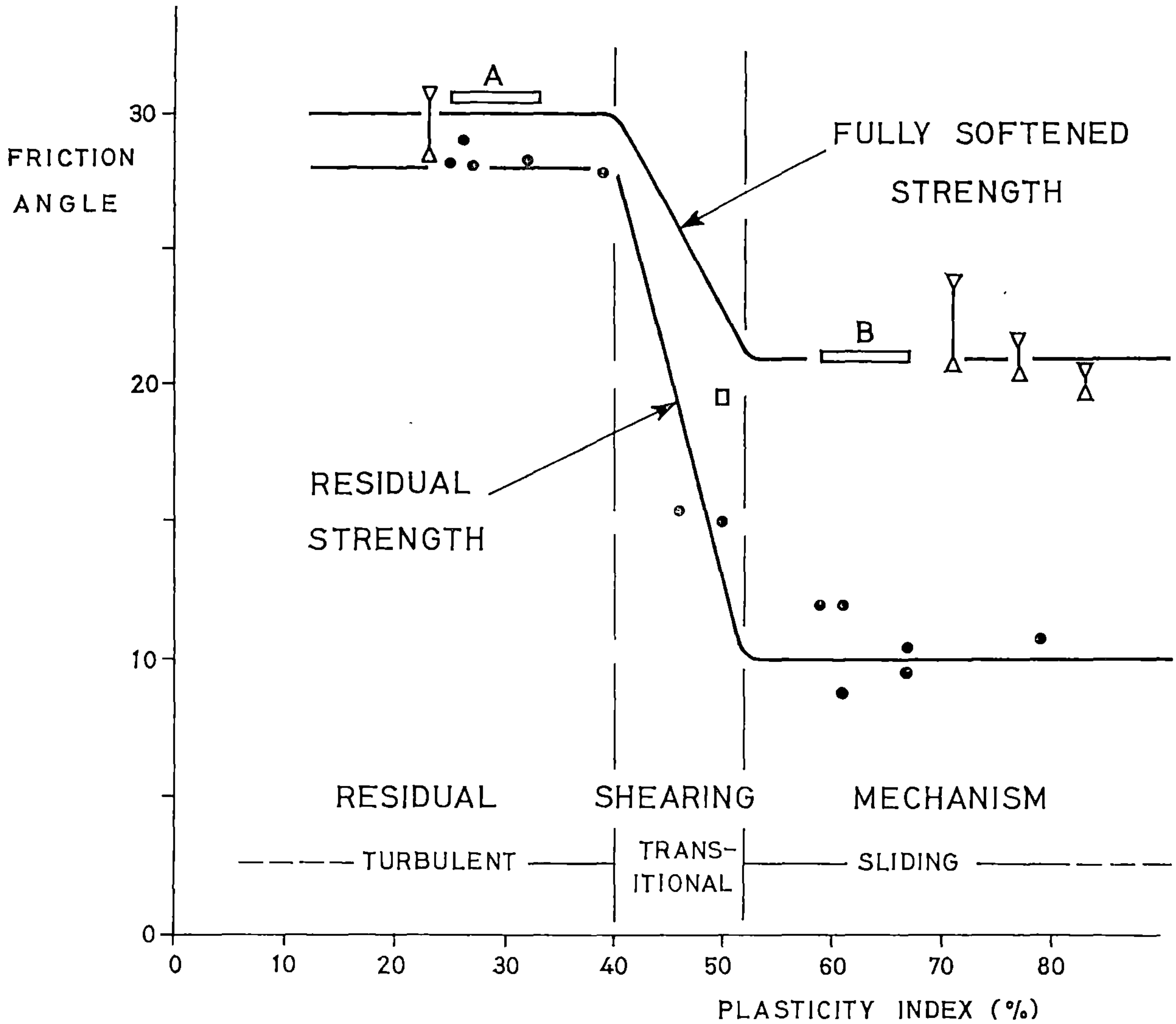
R^2 is a measure of the proportion of variation in the data which is explained by the assumption that the regression equation is linear.

Group B represents the analysis of seven samples whose PI ranged from 59 to 67%. All the other results on Figure 11 represent single samples where multi-stage tests have resulted in the determination of separate failure envelopes for each sample.

The solid lines show the general pattern of results. The correlation between the residual angle of friction (ϕ_r') and plasticity index has already been explained by differences in the residual shearing mechanism caused by variations in clay content (Section 5.4.3).

The solid line indicating the relationship between the fully softened angle of friction (ϕ') and the plasticity index is less well established but can be justified on the following grounds. Up to a PI of 39% the test results indicate a ϕ' only slightly higher than ϕ_r'. Between a PI of 39% and 59% the only information is one remoulded test result which is likely to give a low estimate of ϕ' because of the curved failure envelope (Section D.6.3, Appendix D). For a PI of 59% and above the three triaxial tests could be interpreted as giving a sloping curve. However, the sample which gave the highest strength was tested at lower cell pressures than the other two samples and this may explain the slightly different results. The post peak shear box tests indicate a consistent strength over the range tested (Table D.4, Appendix D). Lupini *et al.* (1981) tested sand-bentonite mixtures in a ring shear apparatus and found little variation in peak strength for clay fractions between 50 and 90%.

The cohesion, of about 3 kPa, obtained in the residual strength tests did not appear to be dependent on the residual shearing mechanism or the PI (Table 3). The fully softened cohesion parameter is assumed to be similar to the residual cohesion (Section 5.3) and therefore, also independent of the plasticity.



SHEAR BOX TESTS

- ● RESIDUAL STRENGTH
- A ▭ POST PEAK STRENGTH FOR PLASTICITY INDEX RANGE SHOWN
- ◻ REMOULDED STRENGTH

TRIAXIAL TESTS

- ▽ MAXIMUM RATIO OF PRINCIPAL STRESSES
- △ MAXIMUM DIFFERENCE OF PRINCIPAL STRESSES

BOVILLS SLIP

RELATION BETWEEN STRENGTH AND PLASTICITY

FIG.11

A summary of the relationship established between effective shear strength parameters and plasticity index is given in Table 6.

TABLE 6

SHEAR STRENGTH PARAMETERS AND PLASTICITY INDEX

	Plasticity index range (%)					
	Below 40		40 to 52		Above 52	
Parameter	c′ kPa	φ′ deg	c′ kPa	φ′ deg	c′ kPa	φ′ deg
Fully softened	3	30	3	21-30	3	21
Residual	3	28	3	10-28	3	10

The best estimate of the boundary between the middle and upper plasticity range is 52% (Table 5 and Figure 11). The position of this boundary is not well defined and may lie anywhere between 50 and 60%.

CHAPTER SIX

RECENT LANDSLIP MOVEMENTS

6.1 INTRODUCTION

The purpose of this part of the project was to find out as much as possible about the recent site history. The road at the base of the slip was realigned in 1973 causing the slope to be undercut. Information about events between 1973 and 1979 has been obtained from the Devonport City Council, the Tasmanian Department of Main Roads, the landowner Mr W.Y. Bovill, and geologists from the Tasmania Department of Mines. Since 1980 surface movements have been monitored by repeated surveys, and subsurface movements have been monitored by regularly checking the PVC piezometer tubes for any deformation.

In this chapter a summary of the recent site history, including measured movements, is presented. The monitoring systems are described in more detail and some of the results are presented in Appendix G.

6.2 HISTORY OF LANDSLIP MOVEMENT

A summary of the main events affecting Bovills Slip and the movements involved is given in Table 7. The boundaries of the East Slip and West Slip, which partly overlap, are shown in Figure 4.

The first known slip at the site occurred in July 1975 although there may have been slips in the previous two years. The second known slip occurred in June 1977. There was less rain than in 1975 but the colluvium would have been weakened by the earlier movement. Fully softened strength parameters would be appropriate for the first failure in 1975 whereas residual strength parameters would apply to the analysis of the 1977 failure. Both these movements were limited to the eastern part of Bovills Slip which is referred to as the East Slip (Section 3.3 and Figure 4).

TABLE 7

RECENT SITE HISTORY

Date	Event	Movement
1973 May - June	Road realignment undercuts slope	?
1975 July	East Slip moves	>1 m
1977 June	East Slip moves, surface regrading, drainage and rockfill at toe	>1 m
1978 August	West Slip moves, drainage and rockfill at toe	>1 m
1979 October	West Slip moves	0.1 to 1 m
1980 May - October	Local movements on West Slip	<20 mm
1981 August	West Slip moves, extends upslope	20 to 30 mm
1982	Dry winter	None

The first movement of the West Slip occurred in August 1978. Fully softened strength parameters would be appropriate in the analysis of the 1978 movement whereas residual parameters would apply to the analysis of all subsequent movements.

After the movement of the East Slip in June 1977 the whole surface was regraded, drainage was installed at the toe of the slip, and the material excavated from the toe area was replaced with rockfill. Since these measures were taken movement of the East Slip has stopped.

Drainage was installed at the toe of the West Slip and the excavated material replaced with rockfill after the movement in August 1978. However, the small movements recorded in 1979, 1980, and 1981 indicate that the West Slip is still close to equilibrium during wet periods and larger movements may occur if there is a very wet winter.

The analysis of some of the events listed in Table 6 is discussed in Section 7.7.2.

CHAPTER SEVEN

SLOPE STABILITY ANALYSIS

7.1 INTRODUCTION

This chapter deals with analysis of the field and laboratory data presented and discussed in earlier chapters. The following topics are considered:

- purpose of analysis
- review of input parameters
- methods of analysis
- model development
- sensitivity analysis
- effects of slope changes caused by recent events and remedial measures

The presentation of these topics involves brief discussion of different aspects of the investigation but overall summaries and conclusions are reserved until Chapter 8.

7.2 PURPOSE OF ANALYSIS

Slope stability analysis may be used for the following purposes:

1. to check the validity of laboratory strength parameters
2. to compare the accuracy of different methods of analysis
3. to check the effects on stability of varying input parameters (sensitivity analysis)
4. to assess the effects on stability of slope modifications and remedial measures (design tool).

For a single case record, items 1 and 2 can only be confidently achieved if the input parameters for the analysis are perfectly known. In the study of natural slopes this is seldom, if ever, the case. Lack of geological detail and lack of piezometer records at the critical time are common problems. Items 1 and 2 are usually attempted when several or many case records are available. The quality of the input parameters available for the analysis of Bovills Slip are reviewed in the following section.

Analysis has been used to investigate items 3 and 4 in the above list. Item 4 has the most practical importance when remedial measures need to be designed for an active landslip and it is often the objective of engineering site investigations of natural slopes.

7.3 REVIEW OF INPUT PARAMETERS

The inputs required for stability analysis have been considered under four general headings (Section 1.3), and the results of the investigations of these topics have been presented in the preceeding chapters. In this section the quality of the data required for analysis is reviewed. More general discussion and conclusions about the investigation are given in Chapter 8.

A summary assessment of the main parameters required for input into stability analysis is given in Table 8 and the assessment is discussed in more detail below. The consequences of errors in the input parameters are considered in Section 7.6.

TABLE 8

REVIEW OF INPUT PARAMETERS

Parameter	Assessment of data	How to improve
SHAPE OF SLIP, GEOLOGY GEOMORPHOLOGY	Fair	Difficult, further drill ing may not help
WATER IN THE SLIP (PORE WATER PRESSURE)	Fair	Continuous monitoring, rainfall intensity
STRENGTH OF SLIP MATERIALS	Good	
MOVEMENT OF THE SLIP	Good	Continuous monitoring. Inclinometers.

As far as the first parameter is concerned the surface and the subsurface shape of the slip has been well defined but there is a problem with the detailed geology. It is known that the failure zone is located entirely within the silty clay colluvium but details of the plasticity variations within the colluvium are not well known (Section F.2, Appendix F). The mode of residual failure and therefore the residual strength is controlled by these local plasticity variations (Chapter 5). Thus it is not known accurately which parts of the slip failed by turbulent shear with a high residual strength and which parts fail by sliding shear with a low residual strength. Although some higher plasticity zones were encountered in the central part of the slip it has not been possible to determine how extensive they are. The deposit is highly variable. It was considered that further subsurface investigations were not warranted as there are not likely to be systematic variations in the plasticity.

As far as water is concerned, it is possible to predict the pore water pressure at the base of the slip for most of the year but peak pressures after high rainfall are much harder to predict accurately. More reliable results could be obtained by continuous monitoring during periods of high rainfall intensity. More responsive piezometers might indicate higher pore water pressure peaks. However, there is sufficient data to suggest that peak pressures at critical times can be estimated to within 2 or 3 kPa over the whole slip (Chapter 4).

The laboratory part of the investigation was successful in that results have been obtained for the effective shear strength parameters of the colluvium. Both the residual and fully softened strength parameters showed a pattern of dependence on the plasticity (Chapter 5).

The investigation of movement has also been successful. Information is available on four slip movements prior to 1980 and since then monitoring has picked up small movements at the surface and the base of the slip. Continuous recording of surface movement by monitoring devices and inclinometers could provide more details on the time and rate of movements.

7.4 METHODS OF ANALYSIS

As stated in Chapter 4, the analysis of the long term stability of natural slopes or cuttings should be carried out in terms of effective stress. Simons and Menzies (1978) demonstrate clearly how the use of undrained shear strengths in a total stress analysis results in completely unreliable factors of safety. All the analytical methods described below involve the use of effective stresses as opposed to total stresses.

Two-dimensional limit equilibrium methods of stability analysis have been used for this project. Three dimensional analyses were considered unnecessary, as side shearing at Bovills Slip is likely to increase the shearing resistance by less than 5% (Chandler, 1976). Consideration of side effects is more important for slips that are long or are deep compared to their breadth.

Four methods of stability analysis have been used (Table 9).

TABLE 9

METHODS OF ANALYSIS

By hand

Janbu's generalised procedure of slices

Bishop's simplified

By computer

Program SLOPE (Bishop's simplified)

Program STABL (Carter's method - modified
Bishop's for general shape)

Janbu's generalised procedure of slices was used to help develop the model. It satisfies all conditions of equilibrium, fits any shape, and can be done by hand (Janbu, 1973). Bishop's simplified method by hand was found to be the quickest and easiest method to use to investigate the effects of slope modifications and remedial measures (Bishop, 1955).

The two computer methods were used for sensitivity analysis. Program SLOPE was written by B.F. Cousins at the University of Tasmania. It is based on Bishop's simplified method and can only be used for circular failures. Program STABL (Siegel, 1975*a*) is based on Carter's method which is a modification of Bishop's method suitable for any shape (Carter, 1971). It does not satisfy all conditions of equilibrium and usually gives conservative results compared with more rigorous methods of analysis (Siegel, 1975*b*).

Many authors have compared different methods of stability analysis and the general conclusion is that Bishop's simplified method invariably produces results comparable with more rigorous solutions (Parton, 1974; Siegel, 1975*b*; Sarma, 1979; Duncan and Wright, 1980). Although truly circular slip surfaces may be rare, circular arcs may be fitted to many less regular slip surfaces without undue error.

7.5 MODEL DEVELOPMENT

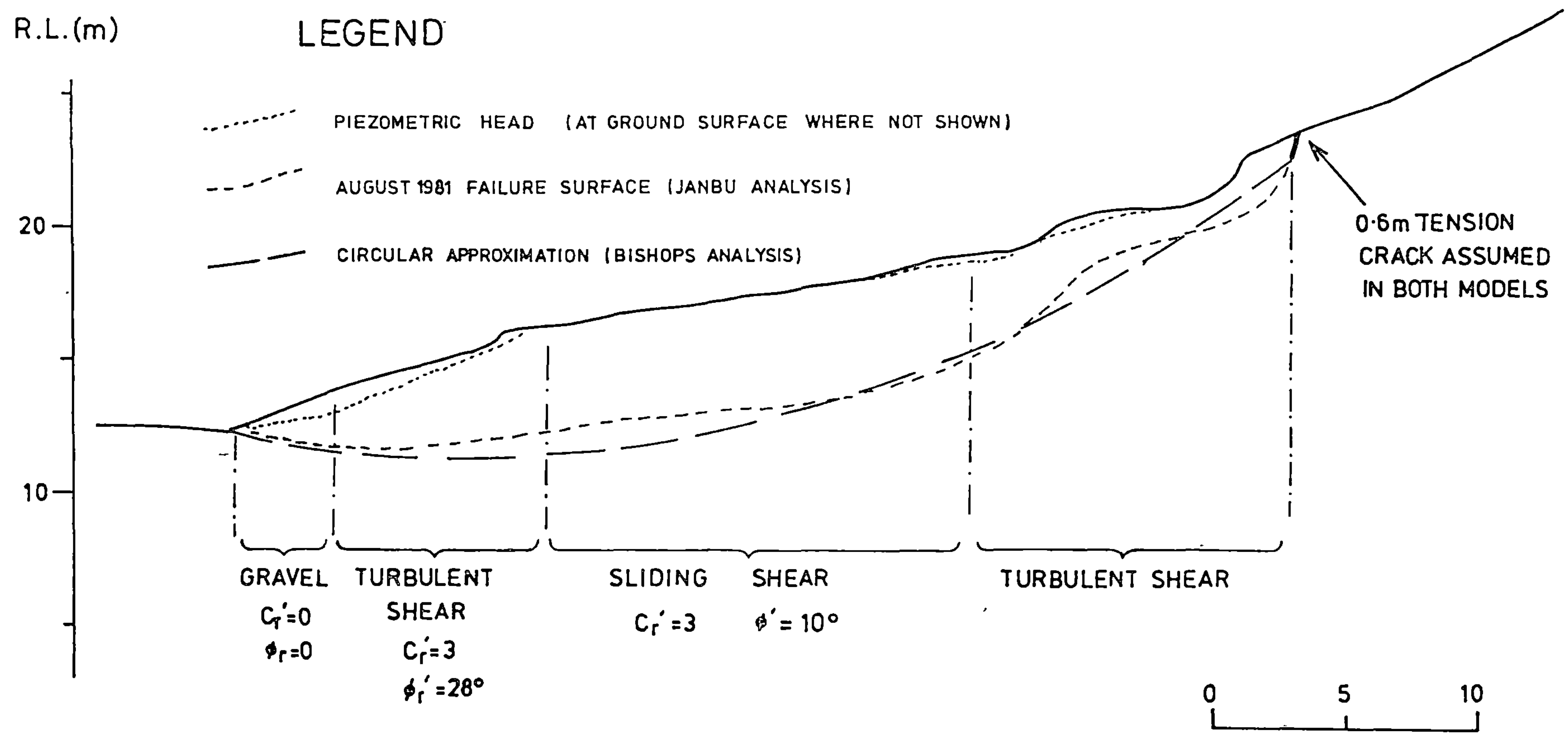
The first model was based on the slope failure of August 1981. This was chosen because the movement observed at that time indicated that the slip was in limiting equilibrium and the factor of safety (F) could be assumed to be 1. The surface shape was taken as the surveyed cross profile along the western grid line (Figure 4). The base of the slip was defined at six points by the observed subsurface movement and was inferred elsewhere from knowledge of the site geology. The pore water pressure at the time of the failure was inferred from measurements before and after movement, and a knowledge of the pattern of pore water pressure variations over a three year period.

Residual shear strength parameters from direct shear tests were available for the silty clay colluvium. In the absence of detailed information it was necessary to make an assumption about the distribution of higher plasticity soil which failed by sliding shear and had a low residual strength (Section 7.3). It was assumed that sliding shear occurred in the central part of the slip as layers and lenses of higher plasticity soil were encountered in the central area. Other parts of the slip were assumed to occur in the lower plasticity soil and fail by turbulent shear with a high residual strength. A 4 m wide gravel drainage layer was assumed to be present at the toe and the rockfill above this layer was assumed to have a similar density to the colluvium. Using Janbu's generalised procedure of slices the width of the central sliding shear part of the model was adjusted until a factor of safety of 1 was obtained. The width of the central part of the model turned out to be 16 m and this figure was used in all subsequent analyses. The final model for the August 1981 failure is shown in Figure 12.

The August 1981 model was also analysed by Bishop's simplified method using a circular arc approximation of the base of the slip. The factor of safety was 1.0 indicating that a model with a circular arc approximation could be used with negligible error. The circular arc is shown in Figure 12.

7.6 SENSITIVITY ANALYSIS

The only inputs into the August 1981 analysis known with certainty were the factor of safety which was 1.0 and the ground surface profile which was regularly surveyed. Other inputs, inferred or measured, may be subject to error. A list of some of these inputs is given in Table 10. The best estimate of their actual value and a range that may be considered to include the 95% confidence interval is given.



NOTES: C'_r IS EFFECTIVE RESIDUAL COHESION (kPa)

ϕ'_r IS EFFECTIVE RESIDUAL FRICTION ANGLE.

BOVILLS SLIP

AUGUST 1981 FAILURE MODELS

SECTION AND STRENGTH PARAMETERS

FIG. 12

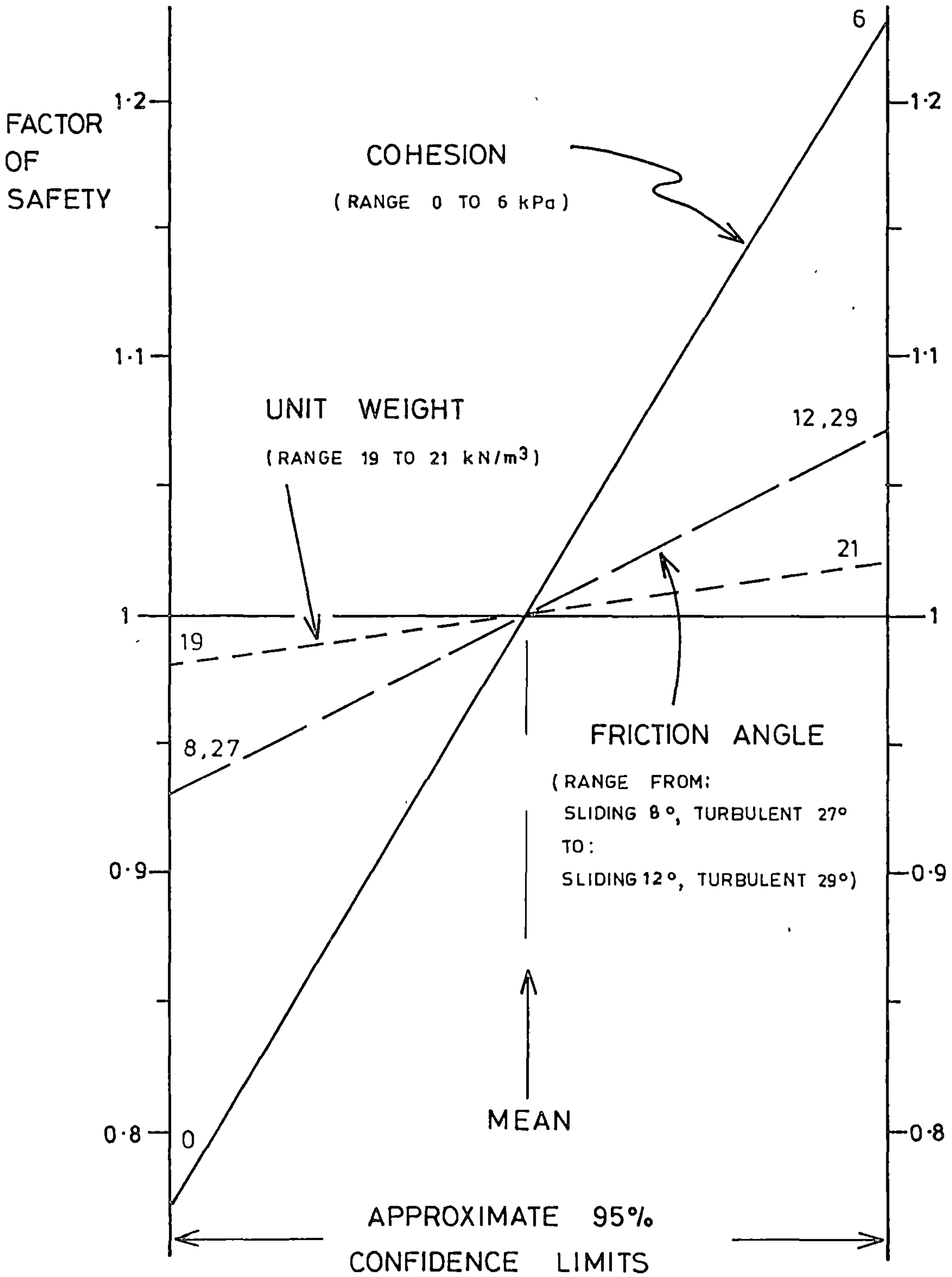
TABLE 10

INPUTS FOR SENSITIVITY ANALYSIS

Input	Unit	Best estimate or mean	Range or 95% confidence interval
Residual cohesion	kPa	3	0 to 6
Residual friction angle - turbulent shear	degrees	28	27 to 29
Residual friction angle - sliding shear	degrees	10	8 to 12
Unit weight	kN/m^3	20	19 to 21

In the case of the strength parameters, the cohesion and the friction angle values given are the actual mean values rounded downwards to the nearest whole number. Similarly actual confidence limits have been rounded downwards or upwards to whole numbers equally spaced from the adopted mean (Section 5.4.3, Table 3). In the case of unit weight the values of best estimate and range are based on density determinations of the soil which have been adjusted slightly to account for the presence of rock fragments (Section F.6, Appendix F).

Program SLOPE was used to carry out sensitivity analyses of the parameters given in Table 10. The effect on the factor of safety of varying the parameters in the given ranges is shown in Figure 13. The central point of the graph represents the starting model where the mean or best estimates of the parameters give a factor of safety of 1. The analysis shows that the factor of safety is most sensitive to changes in cohesion. A cohesion of zero reduces the factor of safety to 0.77 while a cohesion of 6 kPa increases it to 1.23. The analysis is sensitive to variations in cohesion because Bovills Slip is shallow and effective normal stresses are low. The relative effect of the cohesion would be less, and friction would be more for deeper failures. The analysis is



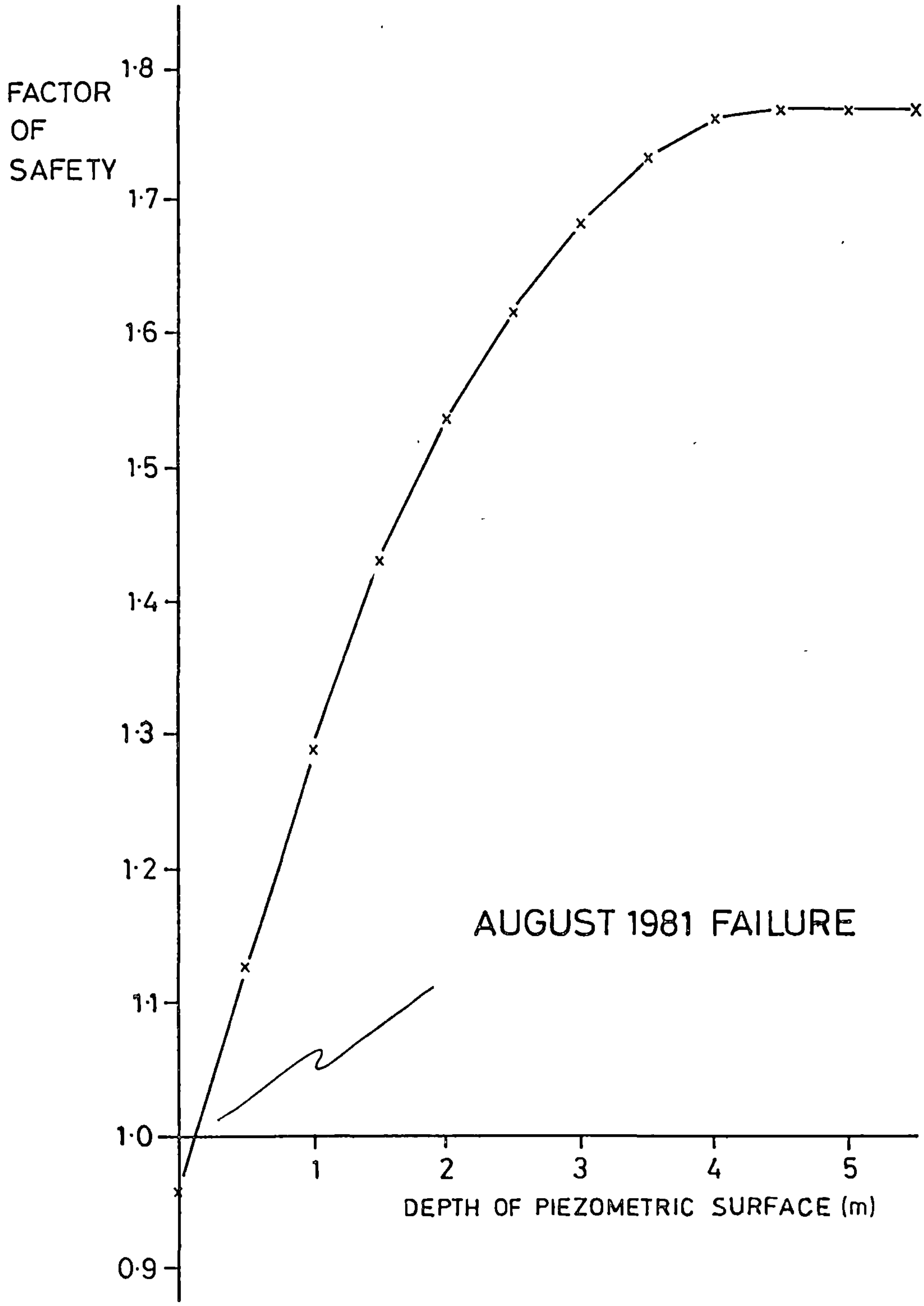
NOTE: PROGRAM SLOPE USED FOR ANALYSIS
PROGRAM STABL GIVES SIMILAR RESULTS

BOVILLS SLIP

SENSITIVITY ANALYSIS

COHESION , FRICTION AND UNIT WEIGHT

FIG. 13



NOTE: PROGRAM STABL USED FOR ANALYSIS

BOVILLS SLIP

SENSITIVITY ANALYSIS

PIEZOMETRIC HEAD

FIG. 14

insensitive to small changes in unit weight. Sensitivity analysis with program STABL produced similar results.

Program STABL was used to determine the effect on the factor of safety of lowering the piezometric surface which reduces the pore pressure on the base of the slip (Figure 14). At the time of the August 1981 failure the average depth of the piezometric surface was about 0.15 m. For most of the year the piezometric surface is more than 2 m deep giving a factor of safety greater than 1.5.

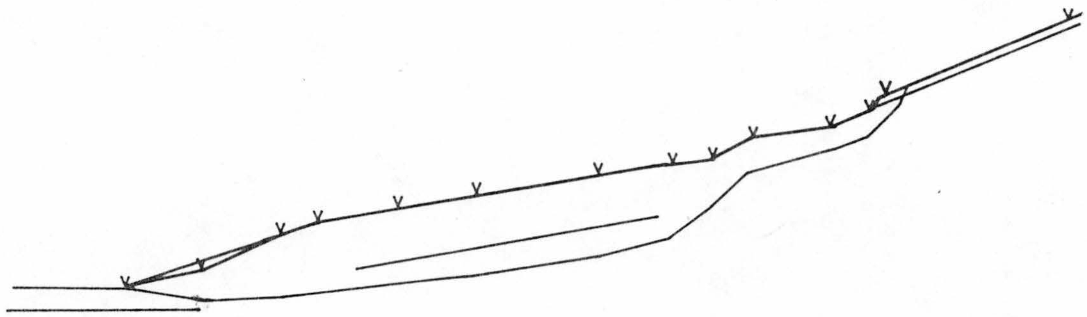
Program STABL was also used to find out whether errors in defining the base of the slip would have any effect on the analysis. A zone, up to 1.6 m wide, known to contain the failure zone was specified and 100 random slip surfaces were generated within this zone. The most critical slip surface had a factor of safety only 1% lower than that used in the model. This indicated that small errors in locating the base of the slip have a negligible effect on the results of the analysis. The slip surface used in the August 1981 model and the zone specified for critical surface search are shown in Figure 15.

7.7 EFFECTS OF SLOPE CHANGES

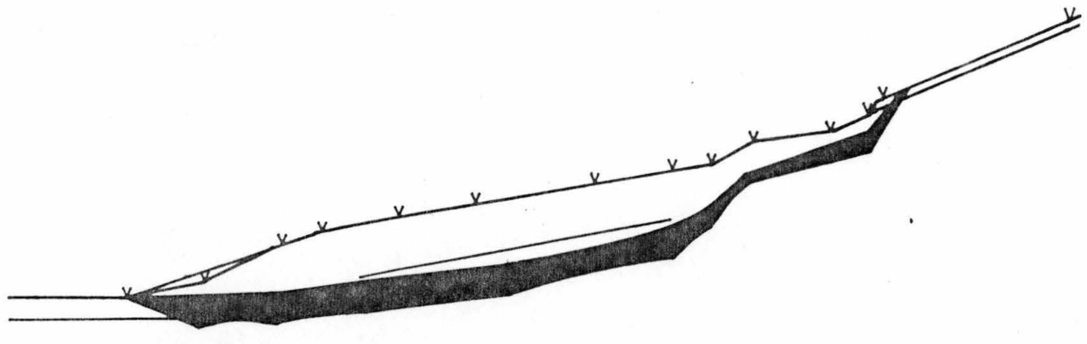
7.7.1 Introduction

Bishop's simplified method of analysis, by hand, has been used to assess the relative change in factor of safety (stability) caused by recent events and by possible future remedial measures. It is emphasised that the analysis involved many assumptions and applies only to Bovills Slip. Similar events or slope modifications at other landslips may cause different effects. The results of the analyses are summarised in Table 11 and discussed in detail in the following sections.

In order to understand their relative effects the different events listed in Table 11 have been analysed separately. In practice, some of the



COMPUTER PLOT SHOWING AUGUST 1981 FAILURE SURACE



COMPUTER PLOT SHOWING ZONE SPECIFIED FOR

CRITICAL SURFACE SEARCH

(PROGRAM STABL USED FOR BOTH PLOTS)

BOVILLS SLIP

SENSITIVITY ANALYSIS

SEARCH FOR CRITICAL SURFACE

FIG.15

TABLE 11

EFFECTS OF SLOPE CHANGES

Event number	Event	Percentage change in factor of safety
1	Removing toe of slope - road realignment	-10 to -15
2	First time slip strength change - fully softened to residual parameters.	WEST SLIP, 1978 -20 to -30 (ALL TURBULENT SHEAR -6 to -8 ALL SLIDING SHEAR -40 to -50)
3	1 m downslope movement - shape change	+5 to +10
4	Toe drainage	+3 to +5
5	1 m toe surcharge	4 m WIDE +5 to +10 8 m WIDE +15 to +20
6	Whole slip drainage, lower maximum piezometric head	BY 0.5 m +15 BY 1 m +30
7	Regrade surface, maximum cut or fill of 0.5 m	+10 to +15
8	Plant trees	+50 (COHESION +35 REDUCE HEAD +15 WEIGHT +1 to +2)
9	Lime stabilisation	? (+8 FOR EACH 1 kPa INCREASE IN COHESION)

events would occur together. For example, the first time slip which reduces the available strength of the soil (Event 2) is accompanied by downslope movement which changes the shape of the slip (Event 3). Several remedial measures (Events 4 to 9) might be carried out at the same time. After the movement of the East Slip in June 1977, toe drainage, rockfill placement and regrading were carried out (Section 6.2).

7.7.2 Recent events

The first event analysed was the effect of removing the toe of the slope when the road was realigned in May 1973. This would have reduced the factor of safety by 10 to 15%.

There are no records of slope movements prior to 1973 and it is considered likely that the roadworks in that year were responsible for the development of Bovills Slip. It is possible that a landslip may not have developed at the site if the toe of the slope had not been undercut.

Fully softened strength parameters apply for the first failure but after a metre or two of movement residual strength parameters should be used. The difference between fully softened parameters and residual parameters depends on the mechanism of residual shear. If the soil fails by turbulent shear, the residual strength will only be slightly lower than the fully softened strength whereas if the soil fails by sliding shear the residual strength is likely to be much less than the fully softened strength (Section 5.6). In the case of Bovills Slip part of the soil failed by turbulent shear and part by sliding shear. The parameter change from fully softened strength to residual strength caused by the first movement of the landslip would have reduced the factor of safety of the West Slip by 20 to 30%. If a landslip consisted entirely of the lower plasticity colluvium which fails by turbulent shear the reduction in factor of safety caused by the parameter change would only have been 6 to 8%. If a landslip consisted entirely of the higher plasticity colluvium which failed by sliding shear the reduction in factor of safety would be 40 to 50%. The significance of the differences in residual shearing mechanisms to the behaviour of landslips is discussed in Section 8.2.4.

Each time a failure occurs the whole slip changes shape and the new shape will have a different factor of safety under similar pore water pressure conditions. The amount of change depends on the curvature of the base of the slip and whether the failed toe is removed. For the West Slip a downslope movement of one metre causes a factor of safety increase of 5 to 10%.

7.7.3 Remedial measures

Toe drainage leads to several changes. The replacement of clay soil by a gravel filter causes a reduction in pore water pressure, an increased friction angle, and a decrease in cohesion. The net result of these changes is to increase the factor of safety of the West Slip by 3 to 5%.

A one metre high rockfill surcharge on the toe is quite effective. If it is 4 m wide the factor of safety increase is 5 to 10%, for a width of 8 m the increase is 15 to 20%.

Surface drainage and subsurface trench drains would have the effect of lowering the maximum piezometric head. Chandler (1977) presents a case record and Hutchinson (1977) presents theory and case records which provide useful information on drainage design. If the maximum piezometric head is lowered by 0.5 m the increase in factor of safety is 15%, for a lowering of one metre the increase is 30%.

Regrading of the surface can improve the stability (Hutchinson, 1977). For a maximum cut or fill of 0.5 m and a total re-arrangement of about 600 m^3 of soil the increase in factor of safety at Bovills Slip would be 10 to 15%.

It is recognised that the clearing of forests can often reduce the stability of slopes (Gray, 1970; Prandini *et al.*, 1977). Conversely, the planting of trees is likely to increase the stability. The increase in stability would occur gradually over many years. It is very difficult to quantify the stabilising effect of trees. Gray (1974) reports three investigations where roots increase the shear strength by increasing the apparent cohesion of the soil. Wu, McKinnell and Swanston (1979) considered that a network of tree roots could increase the soil cohesion by 5 kPa.

They also considered the weight of the trees and the effect on pore water pressures. An increase in cohesion of 5 kPa at Bovills Slip would increase the factor of safety by 35%.

A canopy of trees may also have the effect of reducing the rate at which water enters the ground during periods of intense rain. Foliage in the crown of the trees and organic litter on the forest floor will intercept water before it reaches the ground surface. Evapo-transpiration will also remove water from within the soil. Maximum piezometric heads developed under a forest floor during wet periods are likely to be lower than those developed under open grassland (Prandini *et al.*, 1977). No attempt has been made to quantify this effect at Bovills Slip but if the maximum piezometric head were to be reduced by 0.5 m the factor of safety increases by 15%. Even the weight of the trees has a minor stabilising effect. At the West Slip the increase in disturbing forces caused by the weight of trees is more than compensated by the increase in available strength caused by the higher normal loads acting on the failure zone. Thus the net effect of the tree weight alone is to increase the factor of safety by 1 or 2%. Increases in weight will only contribute to instability in slopes with inclinations above the friction angle of the material involved (Prandini *et al.*, 1977).

In light of the above discussion it appears possible that the effect of well established trees might be to increase the factor of safety at the West Slip by as much as 50%. However, it would take a number of years before trees exert their full effect. Movements of the slip in the meantime could destroy, or slow down the development of, trees in critical areas. Evergreen trees are better than deciduous as evapo-transpiration continues through the critical winter period when slip movements are most likely to occur. Species of *Eucalyptus, Acacia, Melaleuca,* and *Pinus radiata* are all suitable.

It is not possible to predict the precise effect of lime stabilisation. Handy and Williams (1967) report the successful stabilisation of a landslip by quick lime introduced into holes drilled at 1.5 m centres. They report that the lime had migrated a distance of 0.3 m from the drill hole in one year. Lime would be expected to increase the cohesion and may also affect the angle of friction. It is not possible to estimate what the effect on the cohesion would be at the West Slip but for each overall increase in cohesion of 1 kPa there would be an increase in factor of safety of about 8%.

Other remedial measures are reviewed by Hutchinson (1977).

7.7.4 Relative costs of remedial measures

Engineers from the Tasmanian Department of Main Roads have indicated the relative costs of some of the remedial measures. Actual figures were quoted to the writer but they are not reported here as they were indicative only and not based on detailed costings. Relative and actual costs change with time and it would be misleading to apply indicative figures verbally quoted in 1982 for one specific landslip to other landslips at other times.

Regrading, tree planting, and lime stabilisation would be relatively cheap. Toe drainage and toe surcharge combined would be a little more expensive, and drainage of the whole slip with trench drains is likely to be two or three times more expensive than any other alternative.

This discussion of the effects and relative costs of remedial measures should not be taken to imply that further remedial measures are required at the site. The toe drainage and rockfill placed in 1977 and 1978 appear to have been largely effective and since then, as far as the road is concerned, Bovills Slip has only required minor maintenance.

CHAPTER EIGHT

SUMMARY AND CONCLUSIONS

8.1 INTRODUCTION

The primary purpose of this thesis has been to present the results of an investigation of an active landslip and the first part of this final chapter summarises the results of this work. Summaries and conclusions of each aspect of the investigation are presented under headings which represent Chapters 2 to 7 of the main text.

The second part of this chapter presents some ideas for future research on landslips in Tasmania. This section illustrates how the results of the present study may be extended and applied in the future.

8.2 REVIEW OF PRESENT STUDY

8.2.1 Geological setting and geomorphological history

The evolution of the present landscape began during the early part of the Tertiary period when basalt lavas were extruded on to a land surface of Permian sediments and Jurassic dolerite. Throughout the Tertiary period weathering and erosion modified the landscape, and the characteristic red-brown soils were formed on the basalt. In the later part of the Tertiary period a coastal scarp was formed by marine action during a long period when the sea level was similar to or slightly higher than present. At the site of Bovills Slip the coastal scarp is formed on weathered basalt.

During the Quaternary period, colluvium accumulated at the base of the coastal scarp. At the time of the warmest part of the Last Interglacial the sea level in the Devonport area was probably about 20 m above the present level. The colluvium and the weaker weathered basalt at the base of the coastal scarp were removed by wave action in the intertidal zone. The sea level dropped during the Last Glacial Stage and a new deposit of colluvium accumulated at the base of the coastal scarp. During the Holocene the coastal scarp has been relatively stable. Bovills Slip is located in the colluvium that has accumulated at the base of the coastal scarp since the Last Interglacial. The slip was probably caused when the toe of the slope was removed during road realignment in 1973.

8.2.2 Site geology

The colluvium at the base of the coastal scarp is up to 5 m deep and consists of fissured red-brown silty clay with angular rock fragments. Locally there are variations in colour, plasticity and rock fragments.

Bovills Slip is located entirely within the colluvium. The failure zone at the base of the slip coincides with softened zones in the over-consolidated soil.

8.2.3 Pore water pressure and rainfall

Pore water pressures at the site have been measured with open standpipe piezometers.

The pore water pressures showed a correlation with rainfall. Peak pressures occur during the wet winter months and although continuous records were not available there is sufficient data to suggest that pore water pressures at critical times may be estimated to within 2 or 3 kPa.

A predictive model was developed for one piezometer at Bovills Slip which, given the initial pore water pressures and the input of rain, enables prediction of the new pore water pressure. The piezometer chosen was located in a zone of soil the permeability of which provided response characteristics that were judged to indicate the average response of pore water pressure across the whole slip.

Rainfall at any one time is locally quite variable but the use of records from nearby meteorological stations may be expected to provide an estimate of the rainfall on any particular site which is accurate enough for predictive purposes.

8.2.4 Shear strength parameters

Both the residual shear strength and the fully softened shear strength of the colluvium have been investigated by laboratory testing. The residual strength has been investigated by drained multi-stage direct shear tests using a reversing shear box. The fully softened strength has been investigated by several test methods involving both triaxial and shear box apparatus.

The recognition of different residual shearing mechanisms enabled the relationship between effective shear strength parameters and plasticity index to be understood for the colluvium. This was the most interesting new aspect of the research project. As far as the writer is aware this is the first time that the different residual shearing mechanisms have been reported from one natural soil unit. The original work on defining and describing the mechanisms was done with soil mixtures with artificially varied gradings.

If the soil fails by turbulent shear, the difference between the fully softened parameters (appropriate for the analysis of first time slides) and residual parameters (appropriate for the analysis of repeated movements) is small. For soil which fails by sliding shear the difference is large. For soils falling in the transitional zone both strength parameters will be sensitive to small changes in plasticity.

If a slip occurs in soil which fails by turbulent shear, continuous shear planes do not develop, and the residual strength is not likely to be much lower than the fully softened shear strength. Such a slip may stabilise through small changes in geometry or pore water pressure. However, if the soil fails by sliding shear, there will be a large reduction in shear strength and instability may continue, unless remedial action is taken.

Effective strength testing is time consuming and expensive. The amount of testing undertaken for this study represented about fifteen months full time laboratory work and could not be justified in any routine investigation. However, the results presented here indicate how effective strength parameters may be determined with the minimum amount of such testing. Initial work should be aimed at establishing clay mineralogy, grading, and plasticity variations. Residual strength testing with shear box or ring shear apparatus should then be used to determine residual shearing mechanisms and residual shear strength parameters. Once the residual shearing mechanism is established the fully softened parameters may be investigated by either direct shear or triaxial testing.

Geological formations of stiff fissured clay, although varying in grading and plasticity, often have characteristic clay mineralogies. Using the approach suggested above it may be possible to determine a relationship between effective shear strength parameters and plasticity index which will be applicable for a whole region. Investigations of specific cuttings or slopes in such a region need only concentrate on recognising the appropriate shearing mechanism.

8.2.5 Recent site history

Recent site history at the site of Bovills Slip began after road realignment work undercut the base of the slope in 1973. Slip movements have been recorded in most subsequent years. Since 1980 surface movements have been monitored by repeated survey, and subsurface movements have been monitored by regularly checking the PVC piezometer tubes for any deformation.

The first known movement of the East Slip occurred in 1975. Remedial measures taken after further movement in 1977 appear to have stabilised this part of Bovills Slip. The West Slip first moved in 1978 and although remedial action was taken there have been small movements since then.

Early movements of the slip probably amounted to several metres but the largest single movement since monitoring began occurred in August 1981. After a period of heavy rain the West Slip moved downslope by 20 to 30 mm. Larger movements may occur if there is a very wet winter.

8.2.6. Slope stability analysis

A two dimensional model of the August 1981 failure of the West Slip has been analysed by limit equilibrium methods. Analysis has been used to investigate the effects on stability of varying input parameters and to assess the effects on stability of slope modifications and remedial measures.

Confidence in the results of any stability analysis depends on the quality of the input data. A review of the results of the investigation indicates that because of plasticity variations within the colluvium it is not known exactly which parts of the failure zone failed by turbulent shear with a high residual strength and which parts failed by sliding shear with a low residual strength. Data on strength parameters and movement history is good but data on pore water pressure variations could have been improved with continuous monitoring.

Janbu's generalised procedure of slices was used to develop the model, and Bishop's simplified method of analysis by hand was used to investigate the effects of slope modifications and remedial measures. Two computer methods, program SLOPE and program STABL, were used for sensitivity analysis. A comparison of different methods of analysis indicated that a circular arc approximation of the failure zone could be used with negligible error.

Analysis has shown that the factor of safety is most sensitive to variations in the piezometric surface. For most of the year the piezometric surface is more than 2 m deep and the factor of safety is greater than 1.5. The factor of safety is also sensitive to small variations in cohesion but relatively insensitive to changes in angle of friction and unit weight. Small errors in locating the failure zone at the base of the slip have a negligible effect on the factor of safety.

The removal of the toe of the slope when the road was realigned in 1973 reduced the factor of safety by 10 to 15% and was probably responsible for the development of Bovills Slip. The first movements of the slip caused a decrease in available shear strength in the soil. The amount of decrease depends on the residual shearing mechanism as the change from fully softened to residual strength parameters is much greater for sliding shear than it is for turbulent shear. Downslope movements have produced slope changes which have tended to increase the factor of safety.

The relative effect of remedial measures has also been considered. Toe drainage and toe surcharge has already resulted in increased stability. Regrading of the surface would be effective and relatively cheap while subsurface drainage, although effective, would be more expensive. Lime stabilisation and tree planting were also considered. In the long term well established trees may increase the factor of safety by as much as 50%.

8.3 FUTURE RESEARCH

The Department of Mines is not primarily a research organisation but knowledge of the slope failure problem has been built up through regional studies and many individual investigations. This section suggests possible areas of future work based upon what has been learned during this study.

This investigation has been a very detailed study of one active landslip. The next stage would be to investigate a whole region. There are many landslips in basalt-derived soils along the north-west coast and this might be the logical region to consider first. Investigation of other landslips in this region would be very much less detailed than carried out at Bovills Slip. The objective would be to look at many landslips over a wide area and in many cases investigation would be limited to back analysis of failures based on measured profiles but on assumed failure zones and pore water pressures. The purpose of the back analysis would be to determine the field strength of the materials and, in view of the necessity to assume inputs, probabilistic methods would be appropriate. The assumed inputs would be based on data from Bovills Slip and elsewhere. The results of such an analysis might be to indicate that the residual friction angle (ϕ_r') was, for example, in the range 25 to 31°. Such results could be compared with one another and with the actual parameters determined at Bovills Slip.

A general list of questions and related activities which might be considered during the regional study is given in Table 12.

TABLE 12

QUESTIONS AND ACTIVITIES FOR A REGIONAL STUDY

Questions	Activities
Geology?	Geological surface inspections and investigations.
Shape and depth?	Survey profiles, surface mapping, seismic refraction. Test pits and drilling at some sites.
Clay mineralogy?	X-ray diffraction and Atterberg limit tests.
Pore water pressures?	Observe surface seepages and springs which may indicate the piezometric surface. Install and monitor piezometers wherever possible.
Strength parameters? Shearing mechanisms?	Back analysis of failures. Compare Atterberg limits, X-ray diffractions and gradings. Some strength testing.
Movement?	Establish simple monitoring systems wherever possible.
Analysis? Remedial measures?	Carry out stability analysis. The confidence in the input parameters should always be considered. Sensitivity analysis and probabilistic methods are useful in this respect.

In all these activities Bovills Slip could be used as a model against which other data can be compared. Each new observation at any landslip in the region should increase the confidence in subsequent stability analysis undertaken elsewhere. Probabilistic methods provide a method of quantifying this confidence.

A similar approach could be used in the Tamar Valley where there is already a good deal of information on landslips that would permit a regional appraisal. As discussed in Section 8.2.4 it may be possible to establish a relationship between effective shear strength parameters and plasticity index which may be applicable for a whole region.

If the detailed investigation of Bovills Slip is combined with the regional studies suggested above they should lead to an increased confidence in stability analyses of landslips in different geological situations elsewhere in Tasmania.

APPENDIX A

TEST PIT AND BOREHOLE LOGS

A.1 TEST PITS AND BOREHOLES

Two test pits were excavated with a Massey Ferguson backhoe equipped with a 400 mm bucket. Eleven boreholes (1 to 11) were drilled with a trailer mounted Triefus auger drill. Five boreholes (A to E) were drilled with a combination of hand held power auger (Stihl) and hand auger. The locations and depths of the boreholes and test pits are given in Table A.1

TABLE A.1

BOREHOLE AND TEST PIT LOCATIONS

Borehole number	Co-ordinates (A.M.G.) Eastings	Northings	R.L.(A.H.D.)	Depth (m)
1	449,740.95	5,441,046.96	19.06	3.48
2	740.69	047.17	19.07	3.76
3	745.02	054.75	17.49	4.47
4	749.23	061.64	16.12	3.80
5	747.58	062.39	16.12	3.95
6	755.42	050.65	16.61	3.02
7	744.12	056.22	17.34	3.99
8	739.07	049.65	18.66	3.86
9	736.73	063.09	16.31	2.57
10	720	069	16.3	1.45
11	719	070	16.2	1.40
A	749.75	066.11	14.86	1.80
B	737.54	042.21	20.70	1.60
C	738.84	041.36	18.48	1.44
D	734.74	035.79	21.84	1.24
E	752.16	049.79	17.72	1.95
Test pit 1	751	050	15.5	3.6
Test pit 2	727	065	15.0	3.1

NOTE: The accuracy of the survey information is indicated by the number of decimal places used in the above table.

A.2 ENGINEERING LOGS

A basic approach to the engineering logging of soils and rocks is given by Moon (1980), and a list of symbols and abbreviations used on the logs is given in Table A.2. Test pit logs are presented in Figures A1 and A2 and borehole logs in Figures A3 to A18.

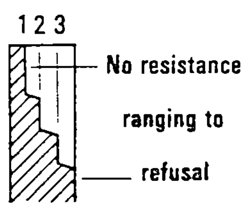
The samples referred to on the logs as U38 were undisturbed samples obtained with standard 38 mm diameter cylindrical sample tubes. Some of these samples were used for triaxial testing. The samples referred to as U70 were collected with sample tubes with a square section 70 mm across. The sample tubes were designed by the writer in order to obtain undisturbed samples suitable for shear box testing.

TABLE A.2

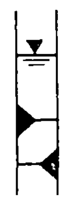
EXPLANATION SHEET FOR ENGINEERING LOGS

Borehole and excavation log

Penetration



Water



22 Jan, 80 Water level on date shown.

Water inflow.

Water outflow.

Notes - samples and tests

U50	Undisturbed sample 50mm diameter
D	Disturbed sample.
N	Standard penetrometer blow count for 300mm.
N *	SPT + sample.

Material classification

Based on Unified Soil Classification System.

In Graphic Log materials are represented by clear contrasting symbols consistent for each project.

Moisture content

D	Dry, looks and feel dry.
M	Moist, no free water on hand when remoulding.
W	Wet, free water on hand when remoulding.
LL	Liquid limit.
PL	Plastic limit.
PI	Plasticity Index.

eg. M > PL - Moist, moisture content greater than the plastic limit.

Consistency

		hand penetrometer (kPa)
VS	Very soft.	< 25
S	Soft.	25 - 50
F	Firm.	50 - 100
St	Stiff.	100 - 200
VSt	Very stiff.	200 - 400
H	Hard.	> 400
Fb	Friable.	

Notes: X on log is test result
▬ is range of results.

Density index

		%
VL	Very loose.	0 - 15
L	Loose.	15 - 35
MD	Medium dense.	35 - 65
D	Dense.	65 - 85
VD	Very Dense	85 - 100

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – EXCAVATION

excavation no. 1
sheet 1 of 1

project BOVILLS SLIP — location BROOKE STREET, DEVONPORT

co-ordinates (A M G) 449,751 E 5,441,050 N	exposure type Pit	pit commenced 18 Mar 1980, 8·30am
RL 17·7 m A.H.D.	equipment Massey Ferguson backhoe 400 mm bucket	pit completed 18 Mar 1980, 10·00am
excavation dimensions 7m × 0·6m × 3·6m deep	operator H. F. Storay	logged by Alan Moon
		checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L. / depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE					CH	CLAY (70%), high plasticity, red brown and ROCK FRAGMENTS (30%), angular, fresh to slightly weathered, extremely high strength basalt up to 0·5m across	D			Continuous near vertical irregular Fissures
				1			Similar to above except CLAY (90%) and ROCK FRAGMENTS (10%), some fine gravel and trace of charcoal fragments	M			Many fissures generally less than 100mm long
				2			CLAY, mottled yellow brown and grey				WEATHERED BASALT COLLUVIUM
				3			CLAY (90%), red brown, ROCK FRAGMENTS (10%)				
			small inflow				Similar to above except CLAY is brown	W / M			Fissure surfaces smooth
							END OF PIT, 3·60m, AT LIMIT OF BACKHOE				

sketch

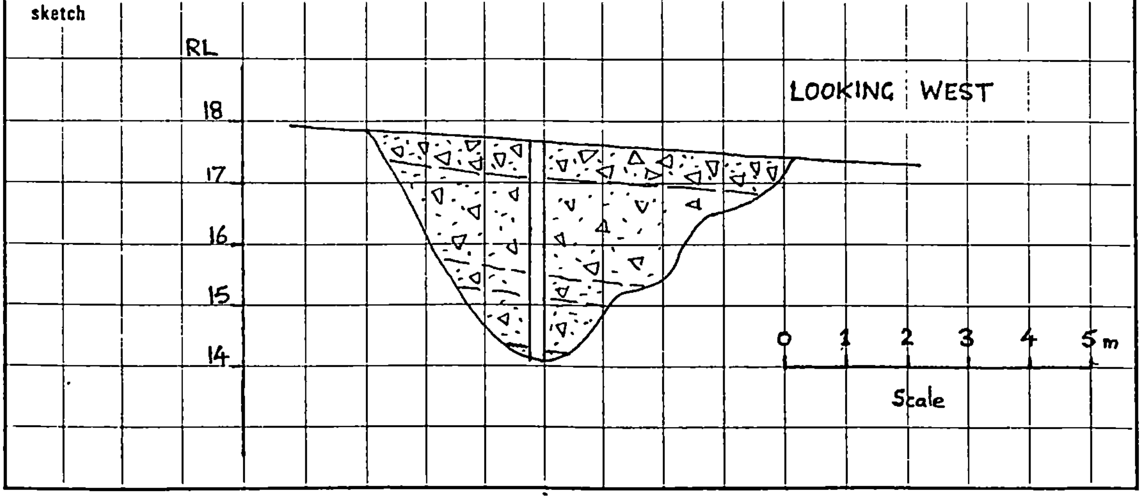


FIG. A1

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – EXCAVATION

excavation no. 2
sheet 1 of 1

project BOVILLS SLIP   location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,727 E 5,441,065 N
R.L 17·2 m A.H.D.
excavation dimensions 6·5m x 0·6m x 3·1m deep

exposure type Pit
equipment Massey Ferguson backhoe 400 mm bucket
operator H. F. Storay

pit commenced 18 Mar 1980, 10·00am
pit completed 18 Mar 1980, 11·00am
logged by Alan Moon
checked by [signature]

penetration 1 2 3	support	water	notes samples, tests	metres R.L.	metres depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE			1		CH	CLAY (90%), high plasticity, red brown, some fine gravel and ROCK FRAGMENTS (10%), angular, fresh to slightly weathered basalt up to 0·3m across, extremely high strength	D			Continuous near vertical irregular fissures
					2, 3			CLAY (90%), similar to above except brown and yellow brown ROCK FRAGMENTS (10%), similar to above	M			Highly fissured. Surfaces smooth and shiny WEATHERED BASALT COLLUVIUM
								END OF PIT AT REQUIRED DEPTH 3·10m				

sketch

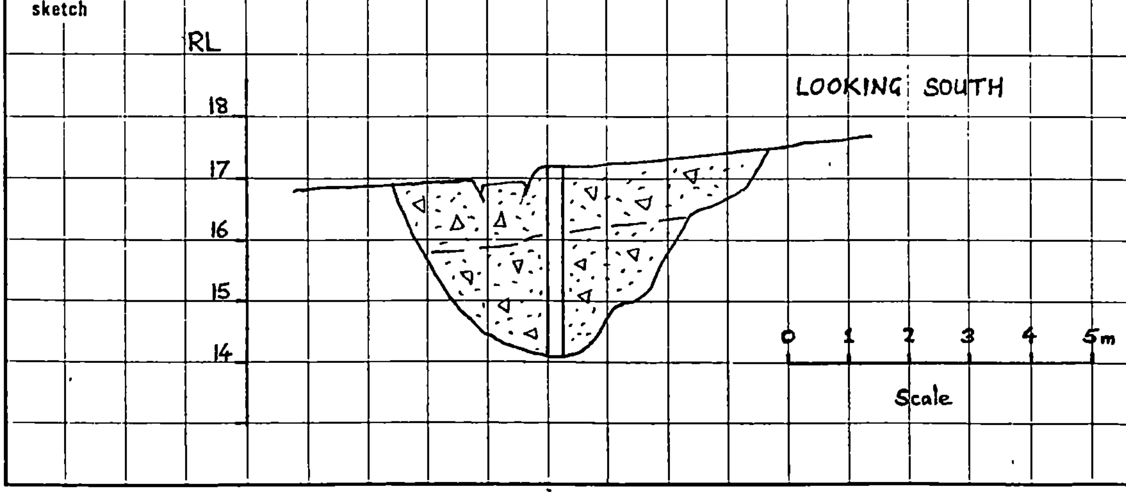


FIG. A2

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no.	1
sheet 1 of 1	

project BOVILLS SLIP — location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,740·95 E 5,441,046·96 N	drill type Triefus	hole commenced 29 Apr 1980, 9·00am
RL 19·06m A.H.D	drill method Auger drilling Tunsten carbide bit	hole completed 29 Apr 1980, 10·30am
inclination vertical	drill fluid None	drilled by Barry Cox
bearing —		logged by Alan Moon
		checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres RL	depth	graphic log	classification symbol	material soil type, plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa (25 50 100 200 400)	structure, geology
	NONE	NONE WHILE DRILLING	D				CH	Silty CLAY, red brown, high plasticity, some sand and gravel (sub angular basalt) and ROCK FRAGMENTS (15%) up to 100mm across	D / M < PL	H		Many fissures
					1			Silty CLAY, similar to above, less ROCK FRAGMENTS (5 to 10%)				WEATHERED BASALT COLLUVIUM
			D									
			U38								>450	
		AFTER 24 HOURS			2							
			D								X X	
									VSt to H			
					3			Gravel content increases with depth				Some EXTREMELY WEATHERED BASALT
			PIEZO D					Gravelly Silty CLAY, red brown, high plasticity				
								END OF HOLE, REFUSAL AT 3·48m				

FIG. A3

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no 2
sheet 1 of 1

project BOVILLS SLIP | location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,740·69 E 5,441,047·17 N
R.L. 19·07m A.H.D.
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling Tunsten carbide bit
drill fluid None

hole commenced 29 Apr 1980, 10·30am
hole completed 29 Apr 1980, 12·00noon
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L. depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE				CH	Silty CLAY, red brown, high plasticity, some sand and gravel (sub angular basalt) and ROCK FRAGMENTS (about 10%)	D	H		Many fissures
							ROCK FRAGMENTS up to 20%				WEATHERED BASALT COLLUVIUM
				1			ROCK FRAGMENTS 5 to 10%				
				2			grading into				
			U70				Silty CLAY, mottled red brown and brown, high plasticity, some sand and gravel		VSt to H	>450	
			D	3							
			PIEZO U38				Gravelly CLAY, mottled yellow brown and brown				
			D				Silty CLAY, mottled red brown and brown				
							END OF HOLE, REFUSAL AT 3·76m				

FIG. A4

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 3
sheet 1 of 1

project BOVILLS SLIP   location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,745·02 E, 5,441,054·75 N
RL 17·49m A.H.D.
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling, Tunsten carbide bit
drill fluid None

hole commenced 29 Apr 1980, 12·00 noon
hole completed 29 Apr 1980, 2·30pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L. / depth	graphic log	classification symbol	material: soil type, plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa (25 50 100 200 400)	structure, geology
	NONE			0		CH	Silty CLAY, dark red brown and red brown, high plasticity, some sand and gravel, with ROCK FRAGMENTS, sub angular basalt up to 50mm (>10%)	D – M < PL	H		Many Fissures
				1							WEATHERED BASALT COLLUVIUM
			D, U38				Less than 10% ROCK FRAGMENTS			>450	
				2							
		AFTER 30 MINUTES	D, U70							>450	
				3				W	St		
			D, U38				Gravelly Silty CLAY, as above	-?- M		>450	
			PIEZO A, PIEZO B	4					VSt		
			D				Silty CLAY, as above				
							END OF HOLE, REFUSAL AT 4·47m				

FIG. A5

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 4
sheet 1 of 1

project BOVILLS SLIP
location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,749·23 E 5,441,061·64 N
RL 16·12m A.H.D.
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling Tunsten carbide bit
drill fluid None

hole commenced 29 Apr 1980, 2:30pm
hole completed 29 Apr 1980, 4:00pm
drilled by Barry Cox
logged by Alan Moon
checked by [signature]

penetration 1 2 3	support	water	notes samples, tests	metres R.L. / depth	graphic log	classification symbol	material soil type plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE			0		CH	Silty CLAY, red brown, high plasticity, some sand and gravel with ROCK FRAGMENTS (< 10%), sub angular basalt	D	H		Many Fissures
								M < PL			WEATHERED BASALT COLLUVIUM
				1							
			D							>450	
			U38							>450	
				2							
		AFTER 1 HOUR	D							>450	
			U70							>450	
				3			Gravelly Silty CLAY, mixture of red brown and dark red brown, high plasticity, gravel consists of weathered basalt fragments	W			Note 'MIXED' materials
			D						St to VSt		
			U38								
			PIEZO D								
							END OF HOLE, REFUSAL AT 3·80m				

FIG. A6

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 5

sheet 1 of 1

project BOVILLS SLIP

location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,747·58 N, 5,441,062·39 E

RL 16·12 A.H.D.

inclination vertical

bearing —

drill type Triefus

drill method Auger drilling, Tunsten carbide bit

drill fluid None

hole commenced 30 Apr 1980, 9·00am

hole completed 30 Apr 1980, 10·30am

drilled by Barry Cox

logged by Alan Moon

checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L. / depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE	D (1.5); D (2.3)	0 – 1 – 2		CH	Silty CLAY, red brown, high plasticity, some sand and gravel with some ROCK FRAGMENTS of sub angular basalt up to 50mm across	D / M < PL	H		Many fissures WEATHERED BASALT COLLUVIUM
			PIEZO B; U38				QUARTZITE PEBBLES recovered between 2·4 and 3·0m, rounded, 10 to 40mm across —— ? ——				—— ? ——
			D; D; U70; PIEZO A; D	3			Silty CLAY – Gravelly Silty CLAY, mixture of red brown and dark red brown, high plasticity, some sand. Gravel consists of slightly weathered to highly weathered basalt, sub rounded to sub angular		VSt to H	>450	Note 'MIXED' materials
							END OF HOLE, REFUSAL AT 3·95m				

FIG. A7

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 6
sheet 1 of 1

project BOVILLS SLIP   location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G.) 449,755·42 E, 5,441,050·65 N
RL 16·61 A.H.D.
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling, Tunsten carbide bit
drill fluid None

hole commenced 30 Apr 1980, 10·30am
hole completed 30 Apr 1980, 11·30am
drilled by Barry Cox
logged by Alan Moon
checked by [signature]

penetration 1 2 3	support	water	notes samples, tests	metres R.L. depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE				CH	Silty CLAY, red brown, high plasticity, some sand and gravel and ROCK FRAGMENTS (about 15%), up to 50mm across	D M < PL	H		Many fissures
			D	1						>450	
				2			Less than 10% ROCK FRAGMENTS				
			D				—— gradational contact ——			>450	
			U38							>450	
			PIEZO D	3			Silty CLAY, brown, high plasticity, some sand and gravel grading down to Silty CLAY, red brown, similar to above		VSt to H	x x	
							END OF HOLE, REFUSAL AT 3·02m				

FIG. A8

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 7
sheet 1 of 1

project BOVILLS SLIP — location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,744·12 E 5,441,056·22 N
RL 17·34m A.H.D
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling Tunsten carbide bit
drill fluid None

hole commenced 30 Apr 1980, 11·30am
hole completed 30 Apr 1980, 1·00pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

notes samples, tests	depth (metres)	classification symbol	material (soil type, plasticity or particle characteristics, colour, secondary and minor components)	moisture condition	consistency density index	hand penetrometer kPa	structure, geology
	0	CH	Silty CLAY, red brown, high plasticity, some sand and gravel with ROCK FRAGMENTS (>10%), sub angular basalt up to 50mm	D	H		Many fissures
D				M < PL		>450	WEATHERED BASALT COLLUVIUM
D	1					>450	
	2		Less than 10% ROCK FRAGMENTS				
D, U38, PIEZO B						>450	
						>450	
	3						
D, U70, PIEZO A			Silty CLAY, red brown, similar to above	W	VSt		
					St		
D				M	VSt		

support: NONE
water: AFTER 30 MINUTES

END OF HOLE, REFUSAL AT 3·99m

FIG. A9

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 8
sheet 1 of 1

project BOVILLS SLIP
location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,739·07 E, 5,441,049·65 N
R.L. 18·66 m A.H.D.
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling, Tunsten carbide bit
drill fluid None

hole commenced 30 Apr 1980, 1·30 pm
hole completed 30 Apr 1980, 2·30 pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L. / depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa (25 50 100 200 400)	structure, geology
	NONE					CH	Silty CLAY, red brown, high plasticity, some sand and gravel with ROCK FRAGMENTS, sub angular basalt up to 50mm (10 to 15% above 0·8m)	D – M < PL	H		Many fissures; WEATHERED BASALT COLLUVIUM
				1			Minor ROCK FRAGMENTS (< 5%)				
			D						VSt to H	X X	
			D	2							
		AFTER 30 MINUTES	PIEZO B, D						VSt	X X	
			D	3			Silty CLAY, red brown, high plasticity, similar to above	W	St to VSt	X X	FAILURE ZONE 3·60 to 3·68m
			PIEZO A, U70			GC	Clayey GRAVEL, red brown, medium plasticity	M	F / VD	X	GRAVEL
			D			CH	Gravelly CLAY, mottled brown, grey, and grey green		St	X	EW BASALT

END OF HOLE, REFUSAL AT 3·86m

FIG. A10

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 9
sheet 1 of 1

project BOVILLS SLIP — location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,736·73 E 5,441,063·09 N	drill type Triefus	hole commenced 30 Apr 1980, 2·30pm
RL 16·31m A.H.D	drill method Auger drilling Tunsten carbide bit	hole completed 30 Apr 1980, 3·30pm
inclination vertical	drill fluid None	drilled by Barry Cox
bearing —		logged by Alan Moon
		checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres RL depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE				CH	Silty CLAY, red brown, high plasticity, some sand and gravel with some ROCK FRAGMENTS	D — M < PL	H		Many Fissures WEATHERED BASALT COLLUVIUM
				1							
			D							>450	
				2							
			PIEZO D U38								
							END OF HOLE, REFUSAL AT 2·57m				

FIG. A11

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 10
sheet 1 of 1

project BOVILLS SLIP — location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,720 E / 5,441.069 N
RL 16·29m A.H.D.
inclination vertical
bearing —

drill type Triefus
drill method Auger drilling
Tunsten carbide bit
drill fluid None

hole commenced 30 Apr 1980, 3·30pm
hole completed 30 Apr 1980, 4·00pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres RL	depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE	D		1		CH	Gravelly Silty CLAY, red brown, high plasticity, some sand and ROCK FRAGMENTS (about 20%), up to 100mm across, weathered sub angular basalt, very high strength	D — M < PL	H	>450	Many Fissures WEATHERED BASALT COLLUVIUM
								END OF HOLE, REFUSAL AT 1·45m				

FIG. A12

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. 11
sheet 1 of 1

project BOVILLS SLIP — location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G)	449,719 E 5,441,070 N	drill type	Triefus
R.L.	16·24m A.H.D.	drill method	Auger drilling Tunsten carbide bit
inclination	vertical	drill fluid	None
bearing	—		
hole commenced	30 Apr 1980, 4·00pm	hole completed	30 Apr 1980, 4·30pm
drilled by	Barry Cox	logged by	Alan Moon
checked by	Alan Moon		

penetration 1 2 3	support	water	notes samples, tests	metres R.L.	metres depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE	D		1		CH	Gravelly Silty CLAY, red brown, high plasticity, some sand and ROCK FRAGMENTS (about 20%) up to 100mm across, weathered, sub angular basalt, very high strength	D M	H	>450	Many fissures WEATHERED BASALT COLLUVIUM
								END OF HOLE, REFUSAL AT 1·40m				

FIG. A13

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. A
sheet 1 of 1

project BOVILLS SLIP   location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,749·75 E 5,441,066·11 N
RL 14·86m A.H.D.
inclination vertical
bearing —

drill type Stihl and hand auger
drill method Auger
drill fluid None

hole commenced 2 Sep 1980, 10·00am
hole completed 2 Sep 1980, 11·00am
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres RL	metres depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE		D		1		CH	Silty CLAY, red brown, high plasticity, some sand and gravel with some ROCK FRAGMENTS (about 10%)	M	H	>450	Many fissures WEATHERED BASALT COLLUVIUM
			PIEZO					Silty CLAY – Gravelly Silty CLAY, mixture of red brown, dark grey, and yellow brown, high plasticity	W M >PL		X X	Note 'MIXED' materials
								END OF HOLE, REFUSAL AT 1·80m				

FIG. A14

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no B
sheet 1 of 1

project BOVILLS SLIP
location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,749·75 E; 5,441,042·21 N
RL 20·70m A.H.D.
inclination vertical
bearing —

drill type Stihl and hand auger
drill method Auger
drill fluid None

hole commenced 2 Sep 1980, 11·30am
hole completed 2 Sep 1980, 12·30pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L.	metres depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE	D				CH	Silty CLAY, dark red brown, high plasticity some sand and gravel and ROCK FRAGMENTS (50%)	M < PL	H		Many fissures WEATHERED BASALT COLLUVIUM
			D		1			Silty CLAY, mixture of red brown and yellow brown, similar to above	M >PL	St to VSt	x x x	
			PIEZO D				SC	Clayey Gravelly SAND, yellow brown, sand fine to coarse, medium plasticity	D	VD		EW BASALT
								END OF HOLE, REFUSAL AT 1·60m				

FIG. A15

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. C
sheet 1 of 1

project BOVILLS SLIP
location BROOKE STREET, DEVONPORT

co-ordinates (AMG) 449,738·84 E 5,441,041·36 N
RL 20·72m A.H.D.
inclination vertical
bearing —

drill type Stihl and hand auger
drill method Auger
drill fluid None

hole commenced 2 Sep 1980, 1·30pm
hole completed 2 Sep 1980, 2·30pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L. depth	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE				CH	Gravelly Silty CLAY, brown, high plasticity, some sand with ROCK FRAGMENTS (about 50%)	M	H		Many Fissures
			D						VSt	X X	
				1			Silty CLAY, similar to above, with pockets of Clayey Gravelly SAND, similar to below				pocket of EW BASALT in COLLUVIUM
			U38					M			
			PIEZO D			SC	Clayey Gravelly SAND, yellow brown, sand fine to coarse, medium plasticity	W D	VD		EW BASALT
							END OF HOLE, REFUSAL AT 1·44m				

FIG. A16

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no D
sheet 1 of 1

project BOVILLS SLIP
location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G) 449,734·74 E 5,441,036·79 N
RL 24·08m A.H.D.
inclination vertical
bearing —

drill type Stihl and hand auger
drill method Auger
drill fluid None

hole commenced 2 Sep 1980, 2·30pm
hole completed 2 Sep 1980, 3·30pm
drilled by Barry Cox
logged by Alan Moon
checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres R.L.	metres depth	graphic log	classification symbol	material soil type plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE		D U38 PIEZO		1		CH	Gravelly Silty CLAY, dark red brown, high plasticity, some sand, with ROCK FRAGMENTS	M < PL	H		Many Fissures
									M > PL	VSt	X	WEATHERED BASALT COLLUVIUM
							SC	Clayey Gravelly SAND, mottled yellow brown, grey, and brown, sand	D	VD		EW BASALT
								END OF HOLE, REFUSAL AT 1·24m				

FIG. A17

TASMANIA DEPARTMENT OF MINES

ENGINEERING LOG – BOREHOLE

borehole no. E
sheet 1 of 1

project BOVILLS SLIP   location BROOKE STREET, DEVONPORT

co-ordinates (A.M.G.) 449,752·16 E 5,441,049·79 N	drill type Stihl and hand auger	hole commenced 3 Sep 1980, 9·30am
RL 17·72m A.H.D.	drill method Auger	hole completed 3 Sep 1980, 11·00am
inclination vertical	drill fluid None	drilled by Barry Cox
bearing —		logged by Alan Moon
		checked by Alan Moon

penetration 1 2 3	support	water	notes samples, tests	metres RL depth	graphic log	classification symbol	material soil type. plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa 25 50 100 200 400	structure, geology
	NONE	NONE	D D D D D D D D PIEZO D D	1		CH	Silty CLAY, red brown, high plasticity, some sand and gravel and ROCK FRAGMENTS	M	H VSt		Many Fissures WEATHERED BASALT COLLUVIUM
							END OF HOLE, REFUSAL AT 1·95m				

FIG. A18

APPENDIX B

SEISMIC REFRACTION RESULTS

B.1 EQUIPMENT AND RESULTS

A seismic refraction traverse was carried out along a cross-section of Bovills Slip (West line, Figure 4) with a SIE RS4 refraction seismograph. Nine shots were fired and the time-distance curves are shown in Figure B1.

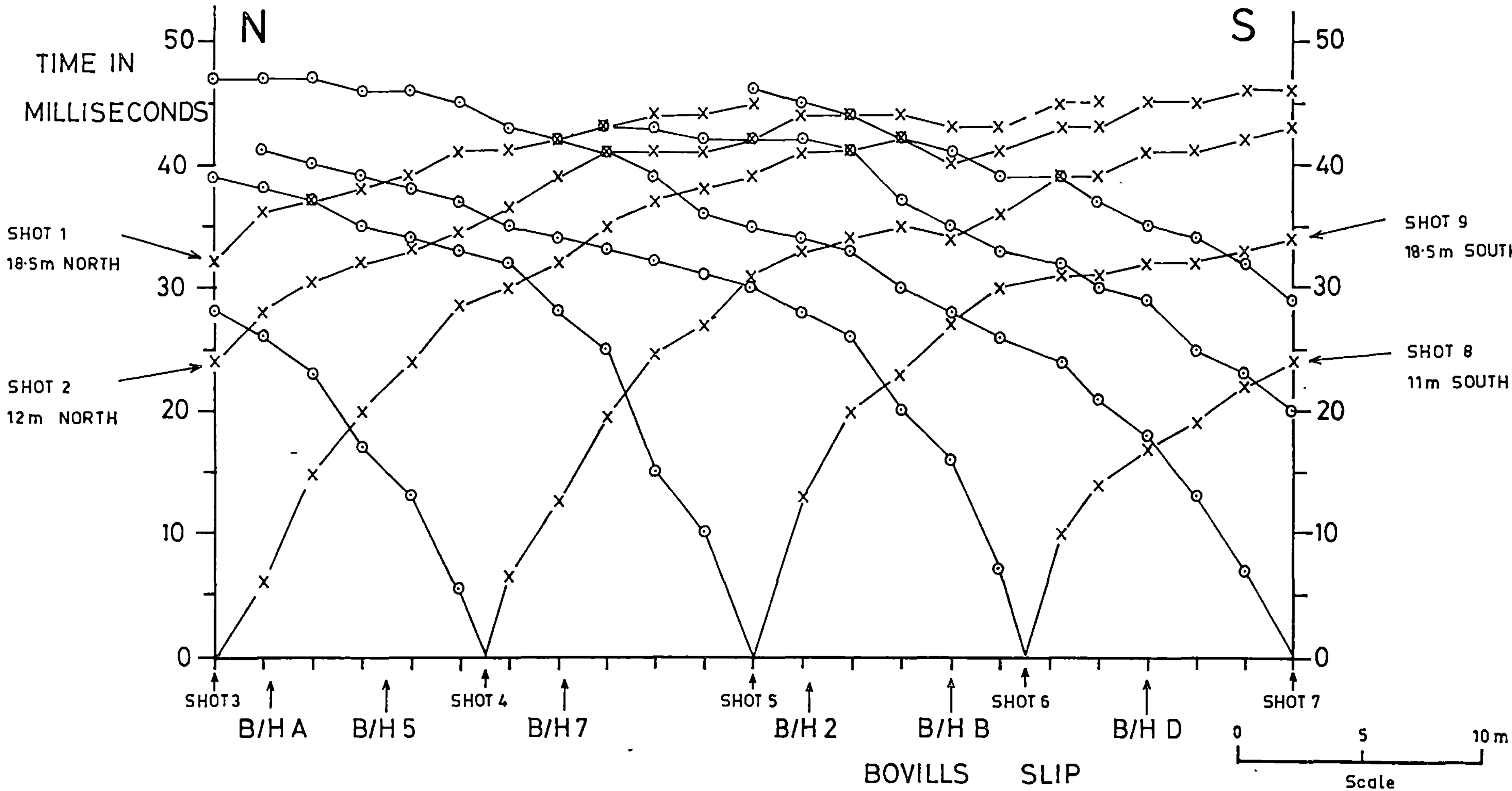
Depth interpretations were carried out by critical distance and reciprocal methods (Hawkins, 1961; Leaman, 1977). The interpreted sub-surface boundaries are shown in Figure 6.

Four layers were detected under the upper part of the slope and three under the lower part. The seismic velocities and interpreted materials are given in Table B.1. The weathering terms used in the table are defined in Moon (1980).

TABLE B.1

SEISMIC VELOCITY AND INTERPRETED MATERIAL

Velocity (m/s)	*Interpreted material*
300 to 450 (150 to 200)	Silty clay colluvium (lower velocity probably represents dry, fissured near-surface material)
700 to 850	Highly to extremely weathered basalt.
1000 to 1200	Slightly to highly weathered basalt.
>2000	Fresh basalt



BOVILLS SLIP

SEISMIC REFRACTION

TIME - DISTANCE CURVES

FIG. B1

NOTES: B/H IS ABBREVIATION FOR BOREHOLE
SECTION ALONG WEST LINE
LOOKING EAST (SEE FIGURES 4 & 6)

APPENDIX C

PORE WATER PRESSURE AND RAINFALL

C.1 INTRODUCTION

This appendix is concerned with the measurement of pore water pressure. As discussed in Chapter 4 it is necessary to know the pore water pressure acting at the base of a landslip in order to carry out effective stress analysis. Pore water pressures at Bovills Slip have been measured with open standpipe piezometers. In this appendix piezometer design is described and the relationship between soil permeability and piezometer response characteristics are discussed.

Pore water pressures vary with time and, at Bovills Slip, rainfall is the main cause of this variation. The measurement of rainfall is discussed. A model which allows prediction of the pore water pressure change for one piezometer caused by a given rainfall input is described. The relationship between pore water pressure and rainfall is also discussed in Chapter 4. Figures are included at the end of this Appendix.

C.2 MEASUREMENT OF PORE WATER PRESSURE

C.2.1 Piezometer design and location

Open standpipe piezometers, as shown in Figure C1, were installed in the auger holes at Bovills Slip. Eighteen piezometers were constructed. Ten auger holes had single piezometers while four deeper holes had two piezometers each. The holes with two piezometers allowed the variation in pore water pressure with depth to be checked. Also, because movement of the landslip could have destroyed the deeper piezometers it was an advantage to have shallower piezometers which may have remained intact.

The depths of the piezometers are given in Table C.1. The piezometers are numbered according to the borehole in which they are located. Where two piezometers are located in the same borehole, the suffixes a and b have been used for the deeper and shallower piezometer respectively.

TABLE C.1

PIEZOMETER LOCATION AND DEPTH

Piezometer number	Depth (m)
1	3.09 to 3.48
2	3.36 to 3.76
3a	4.07 to 4.47
3b	3.65 to 4.00
4	3.40 to 3.80
5a	3.45 to 3.95
5b	2.35 to 2.85
6	2.60 to 3.02
7a	3.50 to 3.99
7b	2.40 to 2.80
8a	3.45 to 3.86
8b	2.35 to 2.75
9	2.25 to 2.57
A	1.40 to 1.80
B	1.25 to 1.60
C	1.15 to 1.44
D	0.80 to 1.24
E	1.70 to 1.95

C.2.2 Permeability and time lag

All the piezometers worked, in the sense that water entered the PVC pipes.

Clearly a certain amount of time is required before rainfall infiltrates the soil and affects the pore water pressure at any point in the failure zone at the base of the slip. Thus the effect of any particular rainfall may be spread over several days. This delay is allowed for in the pore water pressure model described in Section C4.

There is another problem with the measurement of pore water pressure which is dependent on the response characteristics of the particular piezometer. Piezometers take a certain amount of time to respond to changes in pore water pressure in the soil. This is usually referred to as the *response time* or *time lag*. With open standpipe piezometers in low permeability soils there may be a long time lag between a change in pore water pressure in the soil and the corresponding change in pore water pressure in the piezometer cavity. This is because water has to flow into, or out of, the piezometer cavity before a pressure change can be registered. The time lag for pneumatic, hydraulic, and electrical piezometers is very much shorter. Time lag can also be caused by remoulding and smearing of the soil adjacent to the borehole, and by stress changes caused by the drilling of the auger holes and the installation of the piezometers. Leakage up or down the auger hole can also cause problems. The causes and effects of time lag are discussed by Hvorslev (1951), Penman (1960), Gibson (1963), and Vaughan (1974). Another consequence of a long response time or time lag is that a borehole may appear dry when first drilled (Skempton and Henkel, 1960).

The effect of time lag is shown diagrammatically in Figure C2. A piezometer with a short time lag may give a useful approximation of the soil pore water pressure but a piezometer with a long time lag may give quite misleading results.

It is possible to estimate the time lag from permeability tests. Constant head and falling head permeability tests were carried out at 8 of the piezometers. Permeabilities and recovery times were calculated using methods described by Hvorslev (1951). The results of the permeability tests are summarised in Table C.2.

TABLE C.2

TIME LAG AND PERMEABILITY RESULTS

	Piezometer number	90% recovery time	Permeability $\times 10^{-5}$ mm sec $^{-1}$
Shallow piezometers <3 m	5B*	18 hr	2 to 10
	7B	23 sec	4000
	A	70 min	10
	C	230 min	7
Deep piezometers >3 m	1	5 hr	2
	4	25 min	40
	5A	12 hr	1
	8A*	5 hr	1 to 3

* probably some leakage

The 90% recovery time is a measure of the time required for the piezometer to record 90% of an instantaneous change in soil pore water pressure. All the recovery times were less than 24 hours.

The permeability varied quite widely although 6 out of the 8 results were in the range 10^{-4} to 10^{-5} mm/sec. Figure C.3 shows that there tends to be a decrease in permeability with depth. Similar results were obtained by Chandler (1974). Anderson, Hubbard and Kneale (1982) describe an embankment where shrinkage cracks increased the permeability of a clay soil close to the surface. The field permeability is higher than that determined by consolidation tests (Table F.3, Section F) because of the presence of fissures.

C.3 RAINFALL

Rainfall records are available from two recording stations in the Devonport area (Figure 1). The Australian Bureau of Meteorology rain gauge for Devonport is located on the coastal scarp 1.5 km west of Bovills Slip. A rain gauge is also maintained at Devonport Airport, on the coastal plain about 2 km east of the landslip.

The average monthly rainfall for Devonport over a 28 year period and a comparison of 7 years of monthly figures for Devonport and the airport are given in Figure C4. The annual rainfall at the airport is about 15% less than that recorded at Devonport. The daily figures can vary quite widely. For a short period rain gauge records were kept for Bovills Slip. A comparison of the rainfall recorded on the landslip, at the airport, and at Devonport is given in Table C.3.

TABLE C.3

RAINFALL COMPARISON

Date - July 1981	Devonport	Rainfall (mm) Bovills Slip	Airport
26th	6.2	2.1	3.2
27th	5.0	5.1	4.4
28th	0.4	0.4	0.6
29th	0.6	0.4	0.7

Clearly, the only way to determine accurately how much rain falls onto a given area in a given period is to measure it. However, as daily and short term visiting of the site was not possible it was necessary to assume that the official Devonport daily rainfall figures provided an accurate estimate of the rainfall at Bovills Slip.

C.4 PORE WATER PRESSURE MODEL

An attempt has been made to develop a model to predict the variation of pore pressure with rainfall. Given the initial pore pressure and the rainfall the model predicts the new pore pressure for a particular piezometer with given inputs of rain. A model is necessary because of the lack of continuous records from the piezometers.

Figure C5 shows a simple model of the behaviour of water in the colluvium. The colluvium is divided by a system of interconnected fissures.

The 'basement' of highly to extremely weathered basalt is likely to be less permeable than the colluvium and provides a base level for drainage. Without rain, drainage and evaporation will cause a lowering of the piezometric surface towards the base level. With rain, losses still occur but there will be inputs caused by infiltration from above and drainage from upslope.

The model is complicated by the presence of two components of water in the soil. Individual soil structural units (peds) contain water, and water also occurs in the fissures. Evidence for these two components is shown in Figure C6. In the zone between 1.5 m and 2 m summer and winter soil suction values are similar but winter moisture contents are about 5% higher than summer moisture contents. When the summer profiles were measured the piezometric surface was below two metres compared with less than one metre for the winter profile. The winter increase in moisture content shown in the 1.5 to 2 m range may be partly due to water filled fissures. The simple model developed only considers the assumed soil fissure component and does not take into account changes in moisture content in individual soil peds. For this reason it is likely to break down in summer when individual soil peds may not be fully saturated and soil suction forces are high.

Figure C7 shows pore pressure changes predicted by the model for Piezometer 5A for a period in the winter of 1980 compared to actual observations. The model is empirical and the factors used have been derived by fitting curves against actual observations. In the following discussion the figures shown in brackets refer to those used for Piezometer 5A in the example shown in Figure C7. To predict the behaviour of other piezometers different figures would be required.

The model assumes a certain base level for drainage (X = 3 m). Everyday the piezometric head measured from the base level is assumed to drop by a constant percentage (10%, i.e. Drainage Factor K = 0.9). The first 1 mm of every daily rainfall is assumed to be intercepted by vegetation and is ignored. All rainfall in excess of 1 mm is assumed to increase the piezometric head by a certain factor (Infiltration Response Factor, A = 40). Thus in the example shown, if there are 11 mm of rain, 1 mm is ignored and the increase in piezometric head will be 400 mm (40 x 10). The entire increase does not occur on the day that the rainfall is recorded. The effect is spread over several days (½ on first day, 1/3 on second day, 1/6 on third day).

The model can be represented by the following formula:

$$U_1 = K.U_0 + (1 + K)X - A\left(\frac{3P_0 + 2P_1 + P_2}{6}\right)$$

where U_1 = calculated depth of piezometric surface

U_0 = depth of piezometric surface on previous day

K = drainage factor

X = depth to basement

A = infiltration response factor

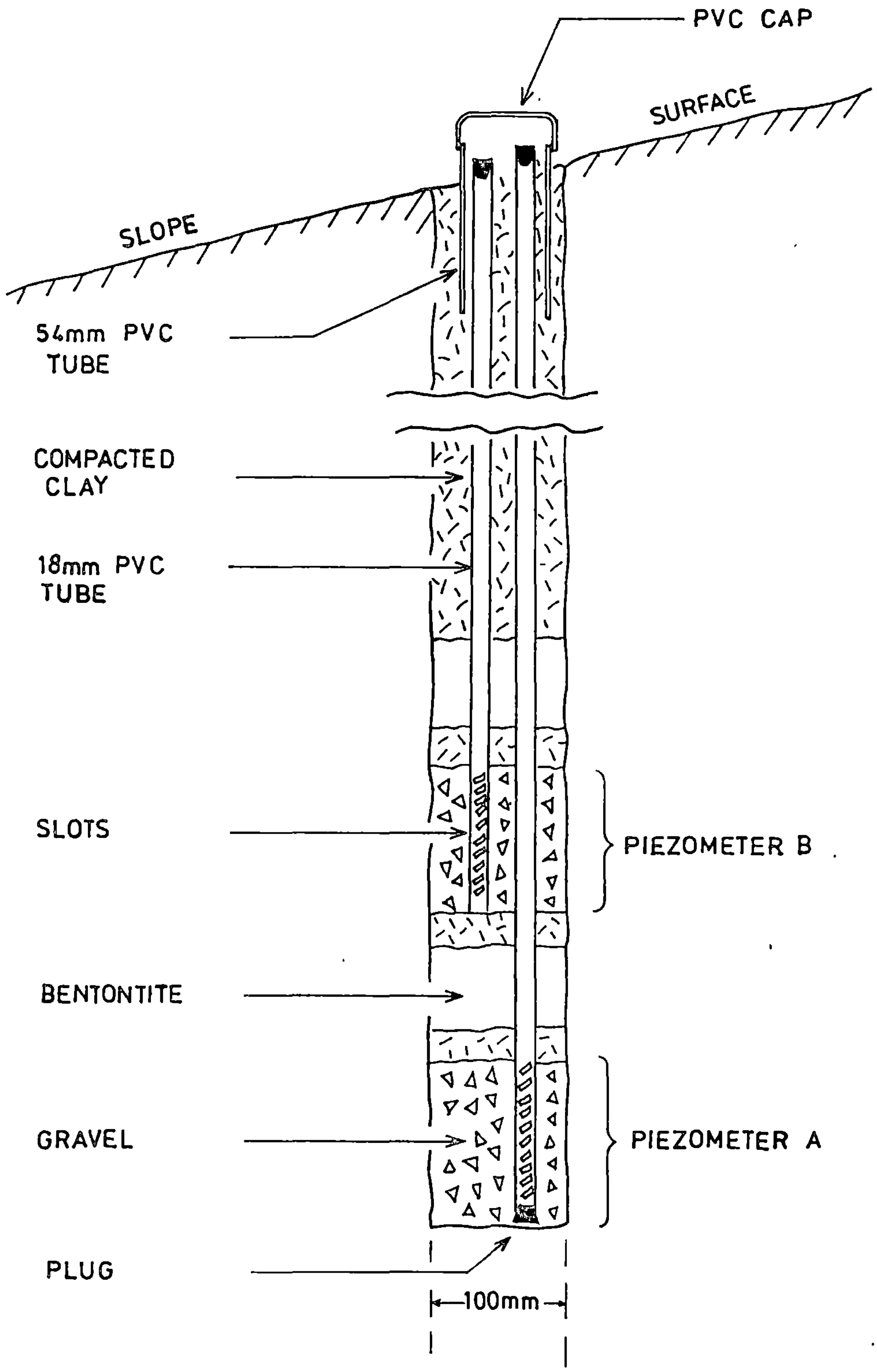
and $\frac{3P_0 + 2P_1 + P_2}{6}$ = rainfall index

where P_0 = rainfall in excess of 1 mm on day for which piezometric head is being calculated.

P_1 = rainfall in excess of 1 mm for day before

P_2 = rainfall in excess of 1 mm for 2 days before

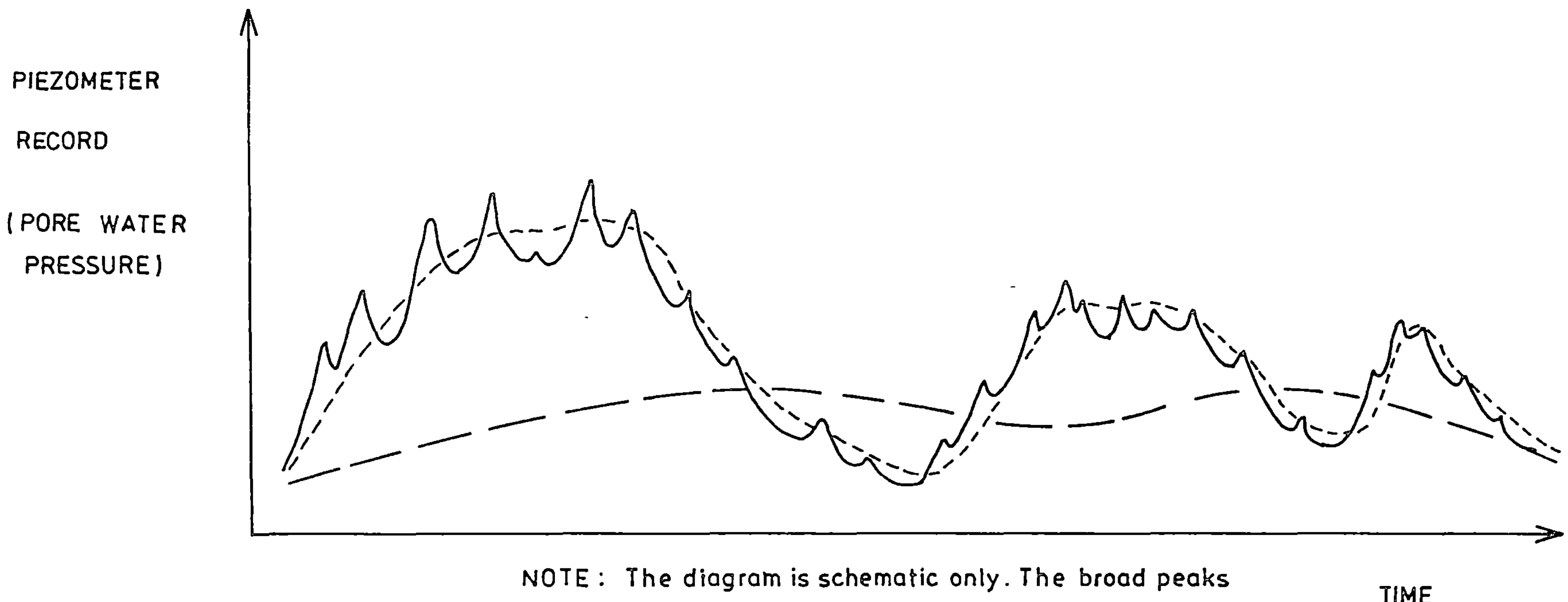
The fact that the effect of any particular rainfall appears to be spread over several days is significant. It indicates that the cumulative effect of a succession of wet days may cause a higher peak in pore water pressure than a large rainfall on a single day. For example, the model predicts that a rainfall of 30 mm on three successive days will increase the pore water pressure more than a single fall of 60 mm. The model was developed for Piezometer 5A because the permeability of the soil around that piezometer was judged to be representative of the permeability of the soil in the whole failure zone at Bovills Slip. Thus the response of Piezometer 5A to rainfall was judged to be a suitable indicator of the general response of the pore water pressure over the whole of the landslip.



BOVILLS SLIP

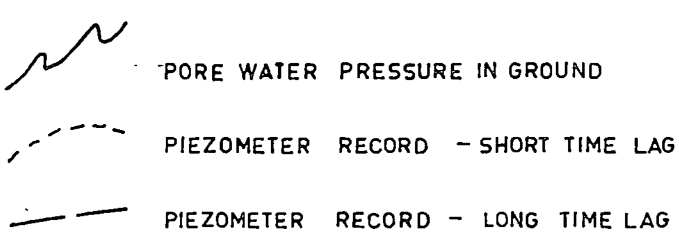
PIEZOMETER DESIGN

FIG. C1



NOTE: The diagram is schematic only. The broad peaks represent pore water pressure during winter.

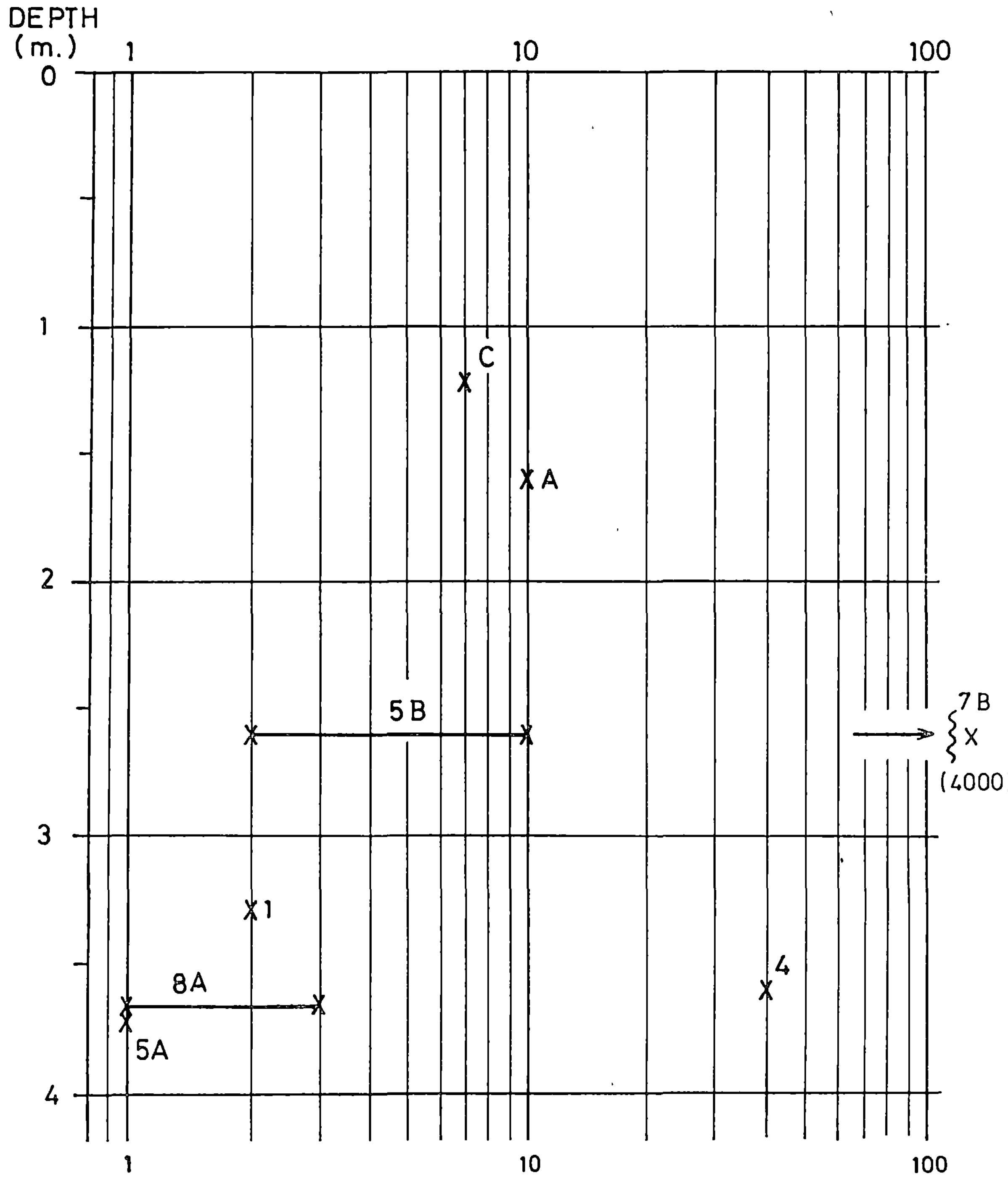
LEGEND



BOVILLS SLIP

EFFECT OF TIME LAG ON PIEZOMETRIC RECORD

FIG. C2



LEGEND

X 5B PERMEABILITY (NUMBER OF LETTER REFERS TO PIEZOMETER)

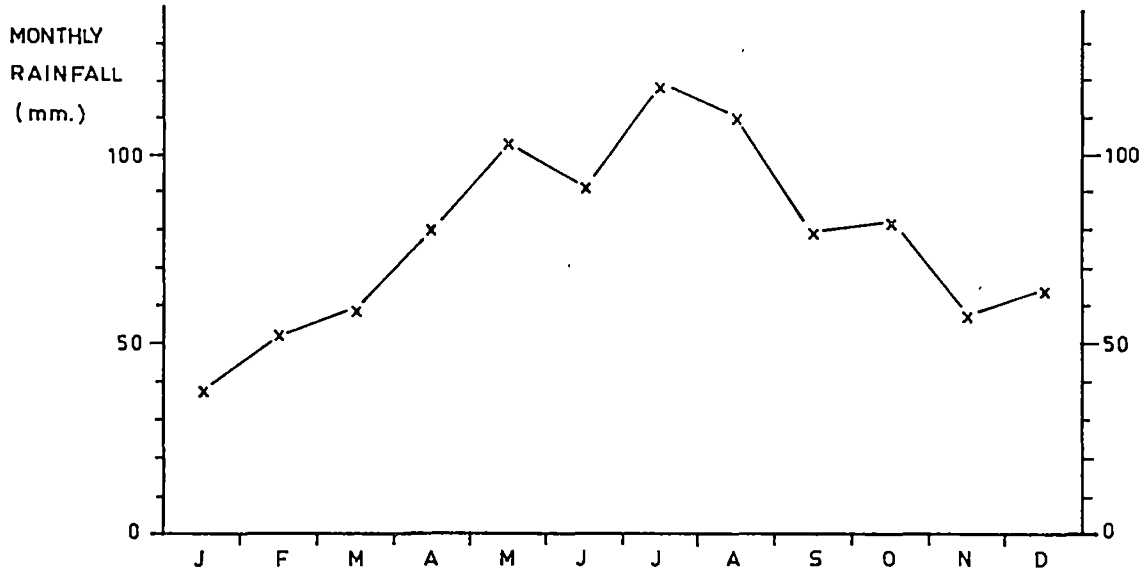
X——X PERMEABILITY RANGE

BOVILLS SLIP

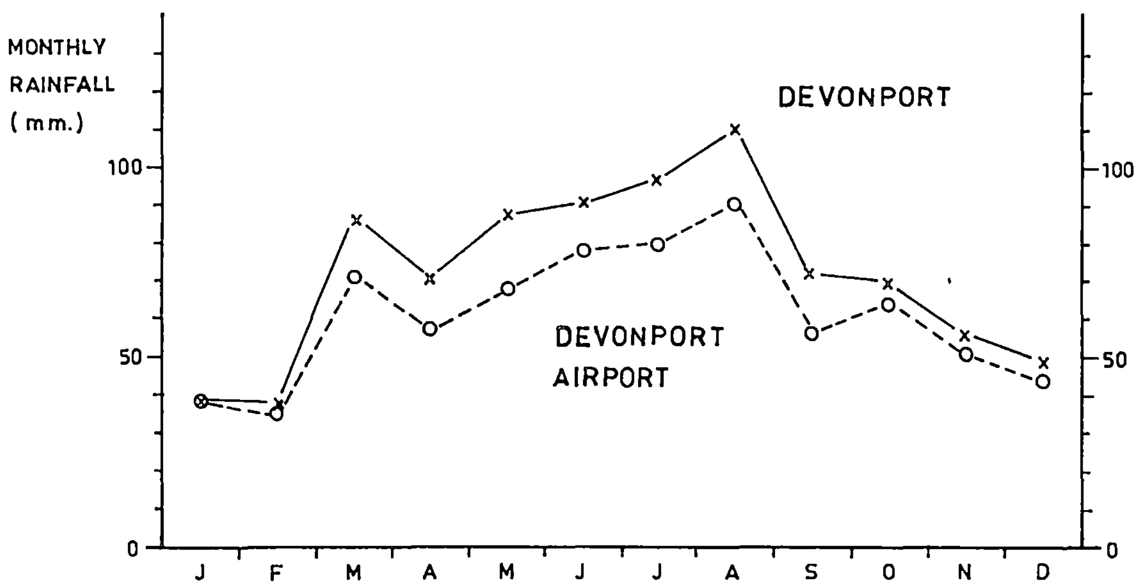
PERMEABILITY

VARIATIONS WITH DEPTH

FIG. C3



AVERAGE MONTHLY RAINFALL AT DEVONPORT, 1954 TO 1982



AVERAGE MONTHLY RAINFAILL AT DEVONPORT AND DEVONPORT AIRPORT 1976 TO 1982

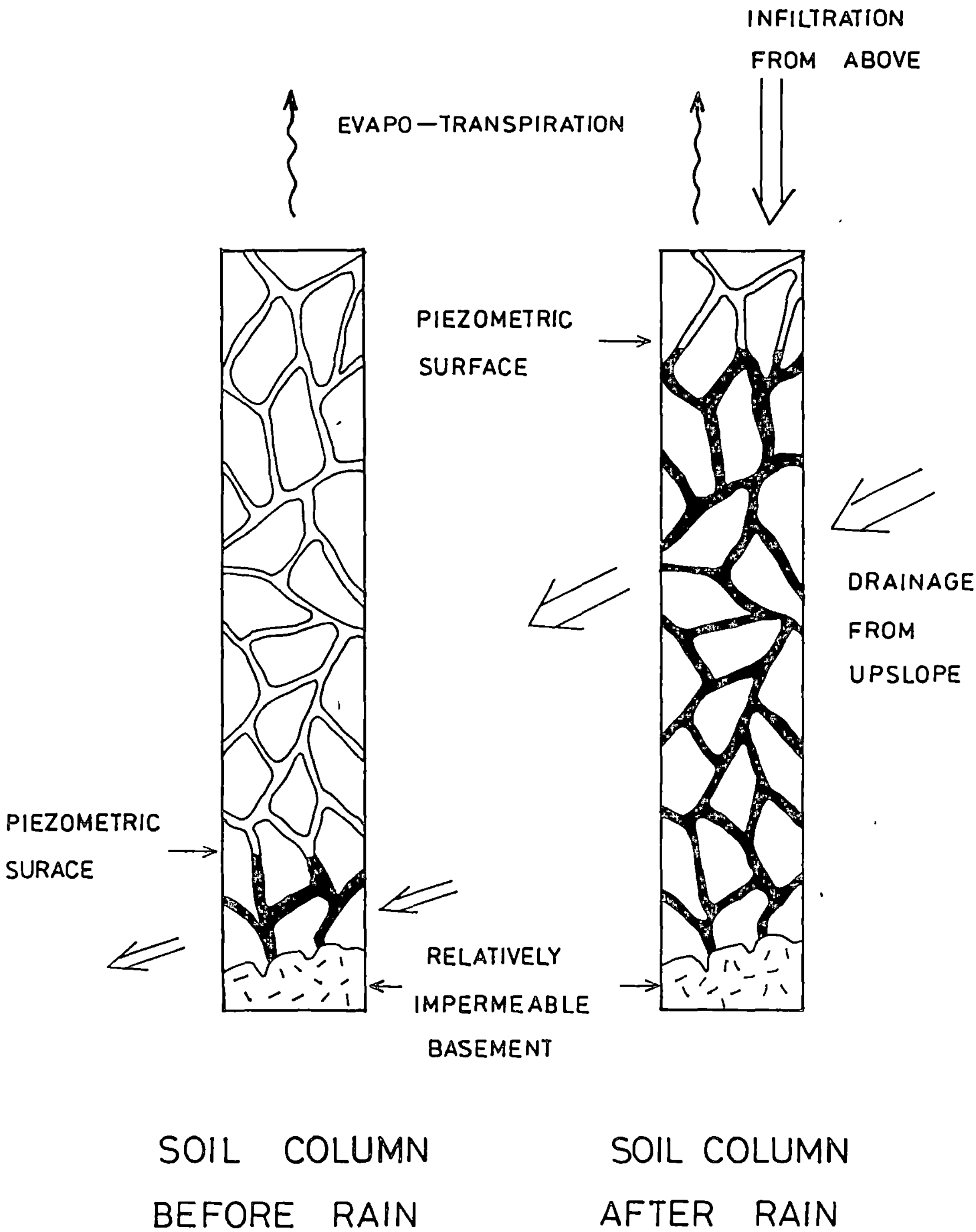
LOCATION OF RAINFALL GAUGES SHOWN ON FIG.1

BOVILLS SLIP

AVERAGE RAINFALL

DEVONPORT AND AIRPORT

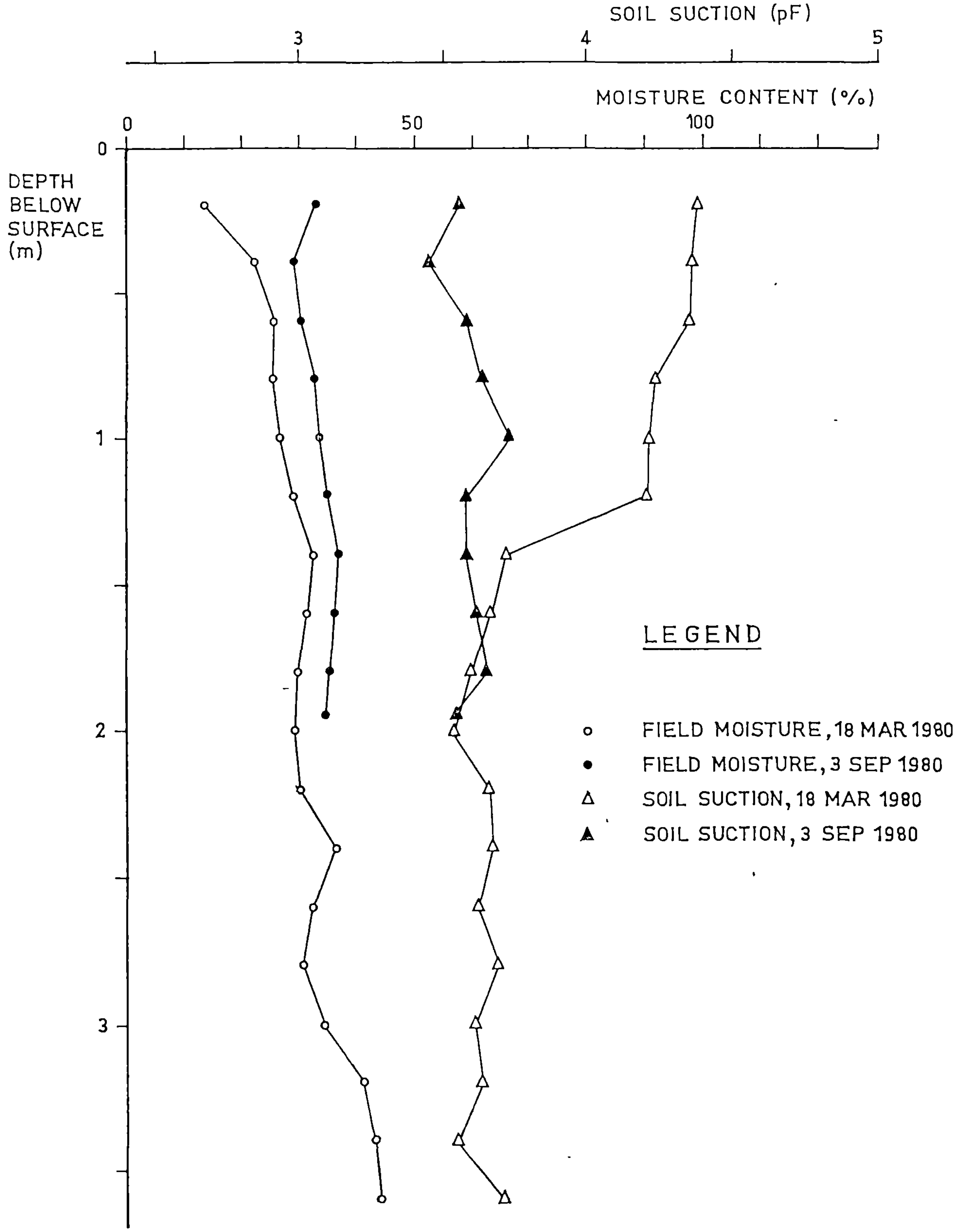
FIG. C4



BOVILLS SLIP

SOIL WATER MODEL

FIG. C5

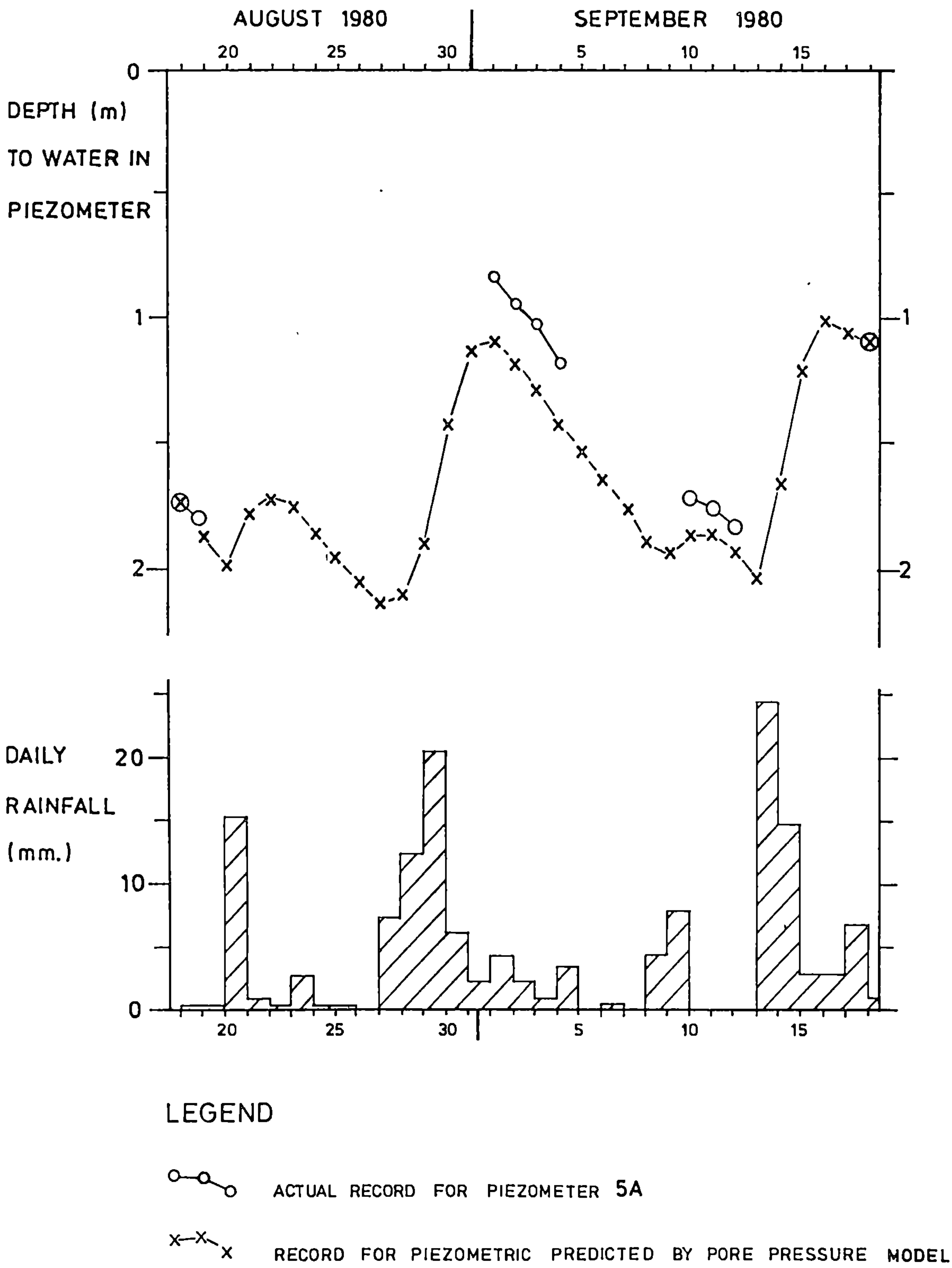


BOVILLS SLIP

TEST PIT 1 EXPLORATION

MOISTURE CONTENT & SOIL SUCTION PROFILE

FIG. C6



BOVILLS SLIP

PORE PRESSURE MODEL

COMPARISON WITH MODEL

FIG. C7

APPENDIX D

SHEAR BOX TESTS

D.1 INTRODUCTION

Shear box tests were carried out in order to determine the residual strength parameters of the silty clay colluvium. Skempton (1964) demonstrated the importance of the concept of residual strength in the long term stability analyses of natural slopes and cuttings in over-consolidated cohesive soils. This appendix includes discussion of different methods of obtaining residual strength parameters, a description of the apparatus used and an account of test procedures. The full results of residual tests on fifteen samples are presented. A summary of the results and a discussion of the relationship of residual strength to other soil parameters is given in Chapter 5.

The shear box was also used to investigate the fully softened strength parameters appropriate for the analysis of first time slides (Chapter 5). The test methods used are discussed and the results are presented. Tables and figures are included at the end of this appendix.

D.2 CHOICE OF TEST TYPE

Residual strength is usually determined from one or more of the following types of test:

reversing shear box
ring shear
triaxial

Most residual strength testing prior to the last 10 years has been carried out in 60 mm square shear boxes (Skempton, 1964; Cullen and Donald, 1971; Chowdhury and Bertoldi, 1977). This apparatus has been found to provide repeatable results for a number of soils. Ring shear tests have become more widely used recently with the development of new apparatus (Bishop *et al.*, 1971). The most significant advantage of the ring shear apparatus is that it allows for large displacements uninterrupted by changes in direction.

Unfortunately, the reversing shear box and the ring shear apparatus sometimes appear to give different results. Bishop *et al.* (1971) report that ring shear tests on Blue London Clay give strengths 30% lower than direct shear tests whereas tests on Cucaracha Shale from Venezuela were up to 15% higher. Chandler *et al.* (1973) report lower strengths from ring shear tests while Townsend and Gilbert (1973) and Newberry and Baker (1981) found that the different test methods gave similar results.

Residual strength parameters have also been obtained from triaxial tests (Chandler, 1966; and Webb, 1969). Experimental difficulties include accuracy at low confining pressures, obtaining sufficient displacement along shear surfaces, and developing corrections for the rubber membrane.

In this project all the residual strength parameters were obtained using a reversing shear box. A ring shear apparatus was not available and a shear box was preferred to triaxial methods because of its comparative simplicity. Results obtained from ring shear tests or triaxial tests may be different from those presented here. The only way to determine the influence of the test method on the results would be by directly comparing the results of tests on similar soils using the different methods.

The most important question is whether the strength parameters determined actually represent the field strength of the materials. In this respect it is instructive to look at the results of laboratory tests and back analyses of other stiff fissured clays. Brown London Clay has been systematically studied for many years. Reversing shear box tests give residual friction angles (ϕ_r') of about 13° while ring shear tests give a ϕ_r' of 8° (Bishop *et al.*, 1971). Observation of natural slopes and back analyses of failures in Brown London Clay suggest that the field ϕ_r' is closer to 13° than to 8° (Hutchinson, 1967; Hutchinson and Gostelow, 1976).

Several back analyses of Liassic Clay in Britain suggest that reversing shear box tests may have over-estimated the residual shear strength while ring shear tests may have under-estimated the strength (Chandler *et al.*, 1973; Chandler, 1976).

D.3 APPARATUS

A standard Engineering Laboratory Equipment Ltd shear box was used for all the tests reported here. A switching system was attached to allow for automatic reversing. A transducer mounted on the proving ring enabled ring deflection to be recorded against time on a chart recorder. The maximum displacement available between the box halves is about 15 mm but the maximum displacement required during the tests was 9 mm. The proving ring operated in compression only so that the shear strength could only be measured during the forward travel of the box.

Calibration tests using uniform rounded quartz sand showed a linear ultimate strength envelope passing through the origin. This indicates that errors associated with frictional resistance in the equipment were negligible.

D.4 TEST PROCEDURES

The shear strength was only recorded during the forward travel of the shear box and the automatic reversing switch was only used as a safety device so that the apparatus could be left unattended. At the end of each forward run the shear box was reversed by hand. This procedure is similar to that described by Chowdhury and Bertoldi (1977). Cullen and Donald (1971) recorded the shear strength in both directions by using a proving ring calibrated in compression and tension. However, they found that the tension and compression loads seldom corresponded exactly, and they continued testing until two consecutive runs in the same direction gave similar results. Thus, although they recorded loads in both directions they only used the test results from one direction.

Multi-stage tests were used as described by Cullen and Donald (1971) and Chowdhury and Bertoldi (1977). Each sample was tested under four different normal pressures consistent with overburden pressure. Test procedures varied slightly but most samples were tested at least twice at each normal pressure to give a more accurate result and to ensure that erosion was not progressively weakening the sample (see Section D.5.2). After each change of normal pressure the sample was left overnight to expand or consolidate before testing continued.

Several different rates of testing were tried in order to work out the maximum rate consistent with fully drained testing. A rate of 0.0047 mm/minute was adopted for the first forward run on undisturbed samples and a rate of 0.0237 mm/minute was used for all subsequent runs on that sample. Rates slower than this gave similar results but faster rates of testing often gave higher strength results, or load displacement curves which were difficult to interpret (Cullen and Donald, 1971).

In most runs the position of the two halves of the shear box was adjusted so that the shear load readings were taken when the two halves were aligned. This avoided the need to consider area corrections. In the first run on an undisturbed sample shear loads are recorded as the box halves move apart and some area correction may seem warranted. However Cullen and Donald (1971) considered this problem and found that area corrections appeared to be unnecessary. No area corrections have been applied to the results presented here.

Handwinding and pre-cutting of failure planes is sometimes used in shear box testing to reduce the time taken to obtain residual values. In some of these tests handwinding was used when the load displacement curves were not flattening. Handwinding did not appear to reduce the time of testing and may have contributed to some sample erosion in the early tests. Pre-cutting of failure planes was not considered as the peak strength of the undisturbed samples was required.

Of the 15 samples tested for residual strength, 10 were undisturbed samples, obtained from 70 mm square section sample tubes. Three tests were carried out on disturbed samples packed in the shear box at roughly field moisture content, and 2 tests were carried out on remoulded normally consolidated samples which were placed in the shear box at a consistency close to the liquid limit. Twenty-six tests of peak and post peak strength were carried out on 23 samples as part of the investigation of fully softened strength parameters. The samples are identified in Table D.1.

D.5 RESIDUAL STRENGTH

D.5.1 Load displacement curves

The form of the load displacement curve obtained depended on the mechanism of residual failure (Section 5.4.2, Chapter 5). Samples which failed by turbulent shear had a high residual shear strength and produced different load displacement curves to samples which failed by sliding shear. Typical load displacement curves for the two types of failure are shown in Figure D1.

For soils failing by turbulent shear the peak strength of an undisturbed sample produced a flat curve which dropped very little during the first forward run. The *peak* shear strength and the *post peak* shear strength (see Section D.6.2) are relevant for the analysis of first time slides. The strengths obtained were compared with estimates of *fully softened* strength obtained from triaxial testing (Chapter 5). Subsequent runs tended to produce flat curves, although in some of the earlier runs the curve continued to rise (Figure D1). For the typical curves the value of the flat section was recorded as the shear strength for that particular forward run. In the case of a continually rising curve, either a value was estimated or no result was recorded.

For soils failing by sliding shear the peak strength of undisturbed samples was usually reached after a 1 mm to 3 mm displacement. There was a marked drop in strength to the *post peak* (7 mm displacement) value. In subsequent runs the shear strength was reduced still further and there was often a small peak at the beginning of each forward test. The flat section of the curve was recorded as the shear strength for each stage.

Two series of tests were carried out on normally consolidated remoulded soil (Section D.6.3). Although the samples subsequently failed by sliding shear the first run in each series produced a flat peaked curve similar to the undisturbed turbulent shear results.

A number of forward runs were required to establish the residual strength at each load. There was a tendency for the load to drop a little from run to run until the residual state was reached. However, the load usually remained approximately constant (flat curve) during each run. After some experimentation it was decided to discontinue each run once the curve was flat and not to continue to an arbitrary displacement. This had the effect of increasing the number of runs that could be achieved each day and reducing the total testing time. In samples failing by sliding shear some of the later runs could be completed after less than 1 mm displacement.

Full records of over 900 load displacement curves are available in files and on chart records in the Department of Mines library. The results presented here (Figures D2 to D16) show the shear load adopted for each forward run. This represents the flat section of each load displacement curve. The amount of forward displacement of the shear box is also shown on the graphs.

D.5.2 Sample erosion

For most of the samples unambiguous residual shear strength results were obtained for four normal pressures after a maximum of about 60 forward runs. However, for the first five samples tested (S3 series) between 75 and 96 forward runs were carried out and sample erosion became a problem. All of these samples failed by sliding shear and developed a continuous polished and slickensided surface. The samples broke easily along this surface when unloaded. Samples failing by turbulent shear did not develop continuous shear surfaces. The results shown in Figures D4 to D8 show that the shear strength was still declining after 70 or 80 runs. Erosion of one corner of the sample S3A was observed on unloading and it was assumed that all of the S3 series were affected.

For these samples it was assumed that erosion caused a small constant percentage reduction in strength with each reversal. The percentage reduction was obtained by fitting curves to the results and varied from 0.1% to 0.7%. In the samples affected by erosion the residual strength adopted was arbitrarily set at the apparent strength after the 20th forward run (except for S3A where the 40th run was used because of very few useable results from the early runs). The sample erosion factor shown in Figures D4 to D8 is given by A in the following equation:

$$\zeta_n = \zeta_m A^{n-m}$$

where ζ_n = shear strength after n runs

ζ_m = shear strength after m runs

and A = sample erosion factor

For example, a sample erosion factor of 0.997 implies that a 0.3% drop in shear strength occurs for each forward run due to sample erosion.

D.5.3 Test results

The values adopted for the residual strength for each sample for each normal load are shown in Figures D2 to D16 and in Table D.2.

Linear regression analyses of the data have resulted in values of effective residual cohesion (C_r') and effective residual friction angles (ϕ_r'). The assumption that the failure envelopes are linear in the range tested is justified by the high values of R^2 (proportion of variation in data explained by linear assumption).

Chandler (1976 and 1977) assumes C_r' is zero, and suggests that residual strength failure envelopes are almost always curved for clays of medium to high plasticity. To some extent the assumption that C_r' is zero leads to curved envelopes. For example, if C_r' is not assumed to be zero the results presented by Chandler (Table 4 and Figure 11 in Chandler, 1976) closely fit a straight line with $C_r' = 2.6$ kPa, $\phi_r' = 9.3°$, and $R^2 = 99.39$.

Values of effective residual cohesion C_r' vary from 1 kPa to 7 kPa and a value of 3 kPa has been adopted for analyses. Lupini, Skinner and Vaughan (1981) report the results of ring shear tests on overconsolidated clays with residual effective cohesion varying from 1 kPa to 6 kPa with an average of about 3 kPa.

D.6 FULLY SOFTENED STRENGTH

D.6.1 Definition and test methods

The definition of fully softened strength and the test methods used to investigate it are discussed in Chapter 5. In this section the shear box test methods are described in more detail. Triaxial test methods are described in Appendix E.

D.6.2 Peak and post peak strength of undisturbed samples

For the first forward run of each shear box test the *peak* strength and the *post peak* strengths have been recorded (Section D.5.1 and Figure D1). The post peak strength has been defined as the strength at the end of the first run which was standardised at a shear box displacement of 7 mm. The box drive rate used for these tests was 0.0047 mm min^{-1}.

It was considered that the failure envelopes defined by the post peak strength would provide a better estimate of the fully softened friction angle. Many of the samples, which were collected in summer, may not have been fully saturated at the start of testing and scatter in the peak strength results could be due to variable increases in effective strength due to negative pore pressures. By the end of the first run (post peak strength), the soil in the failure zone would be likely to be closer to full saturation and negative pore pressures would be less. The results support this argument as the post peak strengths fit linear failure envelopes more closely than the peak strength results (R^3 in Tables D3 and D4).

D.6.3 Peak strength of remoulded samples

A series of shear box tests was carried out on remoulded normally consolidated samples. Remoulded soil with a consistency close to the liquid limit was placed in the shear box and allowed to consolidate overnight before being tested. This process was repeated with consolidation and testing being carried out at four different normal pressures consistent with overburden pressure. The peak angle of friction has been taken as an estimate of the fully softened angle of friction (Table 5, Chapter 5). The relatively low value of R^2 is caused by the slightly curved failure envelope which often results from tests on *young* (i.e. remoulded) soils. This curvature of the failure envelope results in a lower estimate of the angle of friction than that obtained from undisturbed samples.

D.6.4 Test results

The results of the investigation of fully softened strength by shear box testing are given in Tables D3 and D4 and Figures D17 and D18. The results are summarised and discussed in Chapter 5.

TABLE D.1 SHEAR BOX SAMPLES

Sample number	*Test pit or borehole* (TP or BH)	*Depth (m)*	*Sample type* U = undisturbed D = disturbed R = remoulded	*Parameter investigated* R = residual S = fully softened
S1A	TP1	2.21 to 2.24	U	R, S
S1B	TP1	2.24 to 2.27	U	S
S1C	TP1	2.31 to 2.34	U	S
S2A	TP1	3.34 to 3.37	U	R, S
S2B	TP1	3.30 to 3.32	U	S
S2C	TP1	3.37 to 3.39	U	S
S3A	TP1	3.53 to 3.56	U	R, S
S3B	TP1	3.50 to 3.53	U	R, S
S3C	TP1	3.47 to 3.50	U	R, S
S3RA	TP1	3.40 to 3.59	D	R
S3RB	TP1	3.40 to 3.59	D	R
S4A	TP2	2.11 to 2.15	U	R, S
S4C	TP2	2.05 to 2.08	U	S
S5RA	TP2	2.70 to 2.77	D	R
S6A	BH2	2.55 to 2.57	U	S
S9A	BH5	3.51 to 3.54	U	R, S
S10A	BH7	3.44 to 3.47	U	R, S
S10B	BH7	3.47 to 3.50	U	S
S10C	BH7	3.50 to 3.53	U	S
S10D	BH7	3.53 to 3.56	U	S
S11A	BH8	3.65 to 3.68	U	R, S
S11B	BH8	3.68 to 3.71	U	R, S
S11C	BH8	3.74 to 3.77	U	S
S11D	BH8	3.77 to 3.80	U	S
SRA	TP1	3.30 to 3.40	R	R, S
SRB	TP2	1.95 to 2.05	R	R, S

TABLE D.2. RESIDUAL SHEAR STRENGTH RESULTS

Sample number	Residual shear strength (kPa) at effective normal pressure shown				Cohesion, (C_r')		Friction angle (ϕ_r')		R^2(%)	Atterberg limits (%)			Moisture content(%)	
	30.0	57.2	98.1	152.6	mean	95% confidence interval	mean	95% confidence interval		liquid limit	plastic limit	plasticity index	before test	after test
S1A	16.1	31.2	53.7	81.7	0.5	-2.6 to 3.5	28.2	26.7 to 29.5	99.96	53	28	25	28.6	30.8
S9A	19.6	35.4	54.5	86.5	3.5	-3.8 to 10.7	28.3	24.9 to 31.6	99.79	62	30	32	31.0	-
S10A	18.5	34.0	56.5	83.9	3.2	-1.3 to 7.7	28.1	25.9 to 30.1	99.92	59	32	27	29.2	33.4
S11A	22.9	37.4	59.9	87.5	7.3	4.4 to 10.2	27.8	26.5 to 29.2	99.96	72	33	39	40.2	37.8
S11B	20.8	34.7	58.4	88.3	3.7	0.9 to 6.6	29.0	27.7 to 30.3	99.97	57	31	26	38.2	-
S5RA	13.2	21.0	31.9	47.0	5.1	4.2 to 5.9	15.4	14.9 to 15.8	99.99	81	35	46	32.6	42.6
SRA	12.0	20.7	31.5	45.2	4.7	0.9 to 8.5	15.0	12.9 to 17.1	99.77	84	34	50	58.3	45.5
S2A	10.5	16.5	25.0	36.7	4.2	3.6 to 4.7	12.0	11.7 to 12.3	99.99	96	37	59	40.3	48.2
S3A	9.1	15.7	24.0	35.4	3.1	1.2 to 5.0	12.0	10.9 to 13.1	99.91	98	37	61	42.0	36.0
S3B	6.2	10.6	16.2	22.9	2.6	1.5 to 3.7	7.7	7.2 to 8.2	99.88	-			≅42	-
S3C	7.9	12.3	19.2	28.5	2.8	2.3 to 3.2	9.6	9.3 to 9.8	99.99	104	37	67	≅42	-
S4A	9.8	15.3	22.3	33.4	4.1	2.1 to 6.1	10.8	9.7 to 11.9	99.88	118	39	79	40.6	55.0
S3RA	7.1	12.0	18.0	25.4	3.1	0.7 to 5.6	8.4	7.0 to 9.8	99.68	-			≅42	-
S3RB	9.8	15.2	22.4	32.5	4.4	3.5 to 5.4	10.4	9.9 to 11.0	99.97	105	38	67	≅42	54.0
SRB	9.0	14.0	21.2	28.0	4.9	0.6 to 9.3	8.8	6.3 to 11.3	99.11	103	42	61	79.4	60.9

NOTES: Calculations for S3B include a value of shear strength of 32.3 kPa at an effective normal pressure of 220.7 kPa.
R^2 is a measure of the proportion of variation in the data which is explained by the assumption that the regression equation is linear.

TABLE D.3. FULLY SOFTENED STRENGTH - TURBULENT SHEAR

Sample number	*Effective normal pressure (kPa)*	*peak strength (kPa)*	*'post peak' strength (kPa)*
S1A	98.1	73.4	63.4
S1B	152.6	98.0	92.5
S1C	30.0	24.7	22.7
S6A	57.2	32.9	31.9
S9A	98.1	52.8	52.8
S10A	57.2	38.7	36.0
S10B	152.6	92.2	86.4
S10C	30.0	31.0	21.7
S10D	98.1	61.6	49.1
S11A	57.2	39.7	39.7
S11B	98.1	70.5	70.5
S11C	152.6	101.5	101.5

	cohesion (C´)	*friction angle (ϕ´)*	*R^2 (%)*
peak strength	6.5	30.6	99.26
'post peak' strength	2.8	30.4	99.76

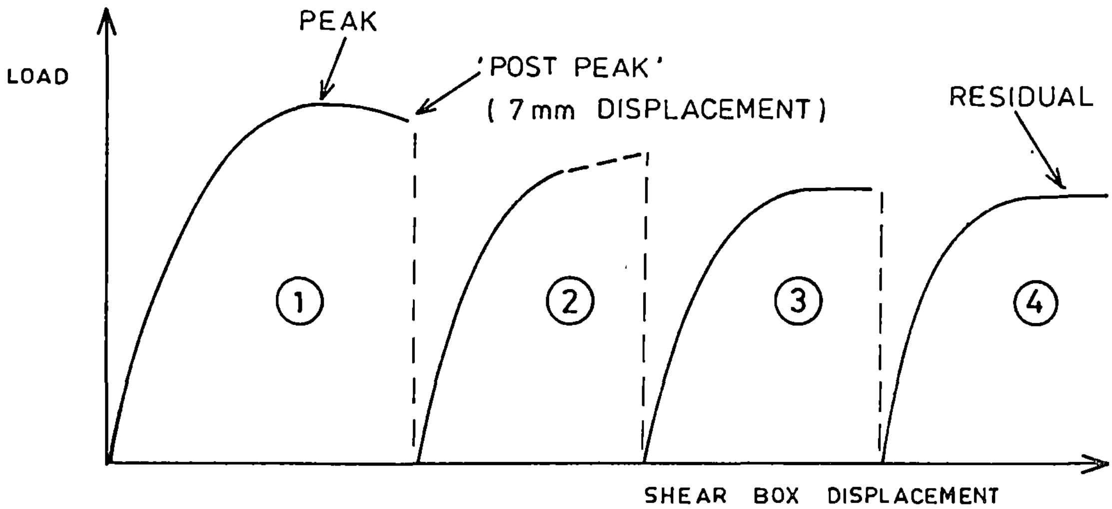
NOTE: R^2 is a measure of the proportion of variation in the data which is explained by the assumption that the regression equation is linear.

TABLE D.4. FULLY SOFTENED STRENGTH - SLIDING SHEAR

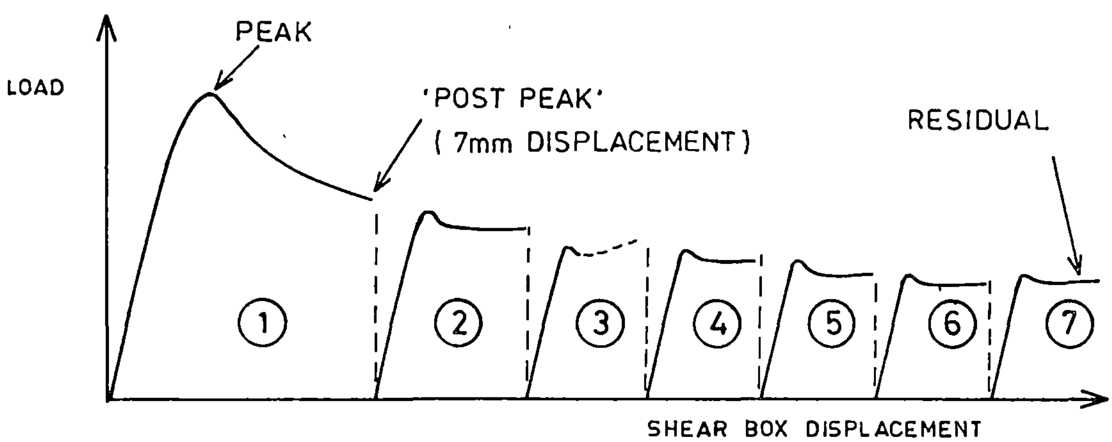
Sample number	*Effective normal pressure (kPa)*	*peak strength (kPa)*	*post peak strength (kPa)*
S2A	98.1	55.0	46.0
S2B	152.6	86.1	68.0
S2C	57.2	42.4	31.1
S3A	30.0	20.8	18.3
S3B	98.1	63.0	40.1
S3C	57.2	50.1	26.0
S4A	98.1	55.9	48.3
S4C	152.6	69.5	63.1
S11D	30.0	24.6	21.2
SRA-1	30.0	15.6	
SRA-3	98.1	42.7	
SRA-4	57.2	28.2	
SRA-5	152.6	59.9	
SRB	152.6	57.5	

	cohesion (C′)	*friction angle (φ′)*	*R^2 (%)*
Peak, undisturbed	15.7	22.9	95.06
Post peak, undisturbed	7.8	20.7	99.91
Remoulded peak (SRA)	6.5	19.6	99.38

NOTE: R^2 is a measure of the proportion of variation in the data which is explained by the assumption that the regression equation is linear.



TURBULENT SHEAR - TYPICAL LOAD DISPLACEMENT CURVE



SLIDING SHEAR - TYPICAL LOAD DISPLACEMENT CURVE

NOTES: NUMBERED STAGES REFER TO SUCCESSIVE FORWARD TESTS

DASHED LINES SHOW CONTINUALLY RISING CURVES
WHERE LOAD VALUE ESTIMATED OR NOT RECORDED

BOVILLS SLIP

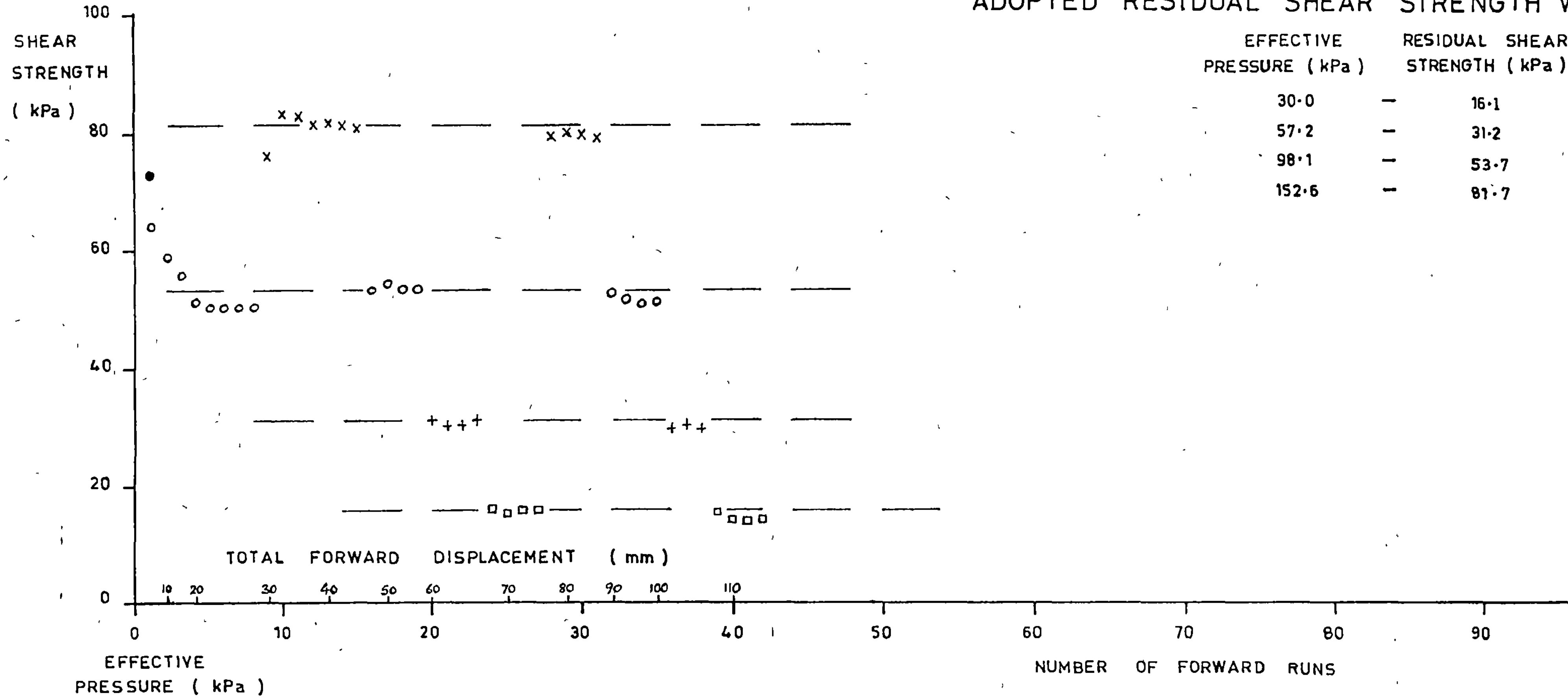
DIRECT SHEAR TESTS

TYPICAL LOAD DISPLACEMENT CURVES

FIG. D1

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	16·1
57·2	—	31·2
98·1	—	53·7
152·6	—	81·7



□ — 30·0
+ — 57·2
o — 98·1
x — 152·6

● — PEAK VALUE

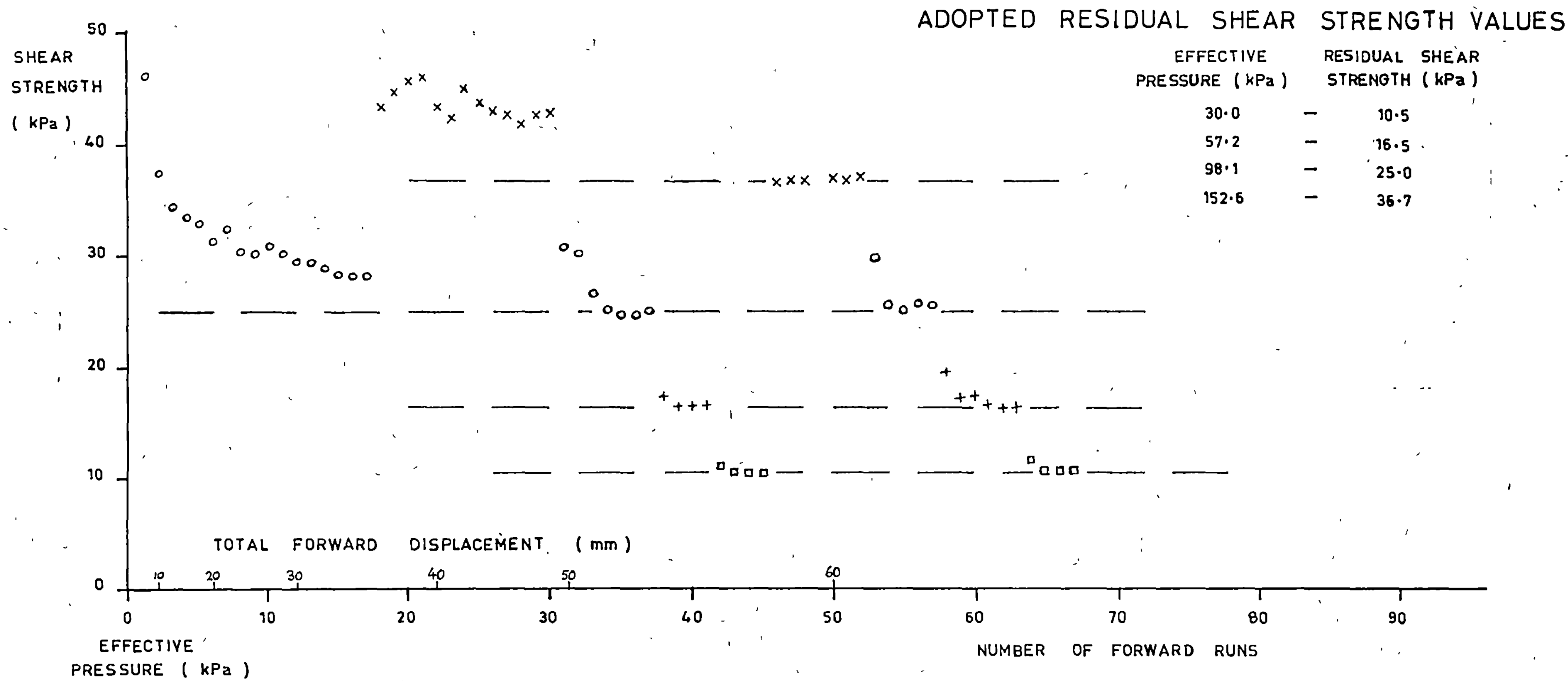
NO SAMPLE EROSION FACTOR

BOVILLS SLIP

DIRECT SHEAR TEST S1A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D2



□ — 30·0
+ — 57·2
o — 98·1
× — 152·6

● — PEAK VALUE

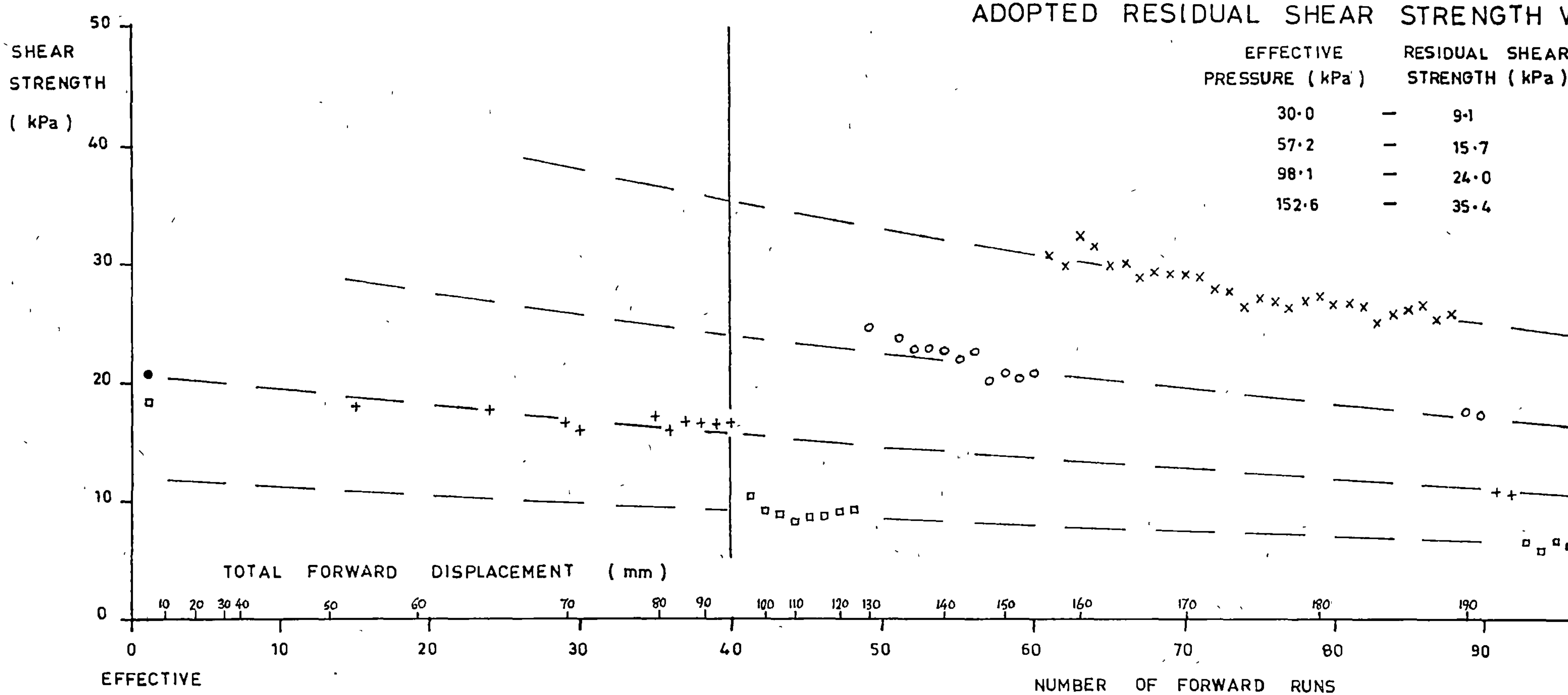
NO SAMPLE EROSION FACTOR

BOVILLS SLIP

DIRECT SHEAR TEST S2A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D3



ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	9·1
57·2	—	15·7
98·1	—	24·0
152·6	—	35·4

EFFECTIVE PRESSURE (kPa)

□ — 30·0
+ — 57·2
o — 98·1
× — 152·6

● — PEAK VALUE

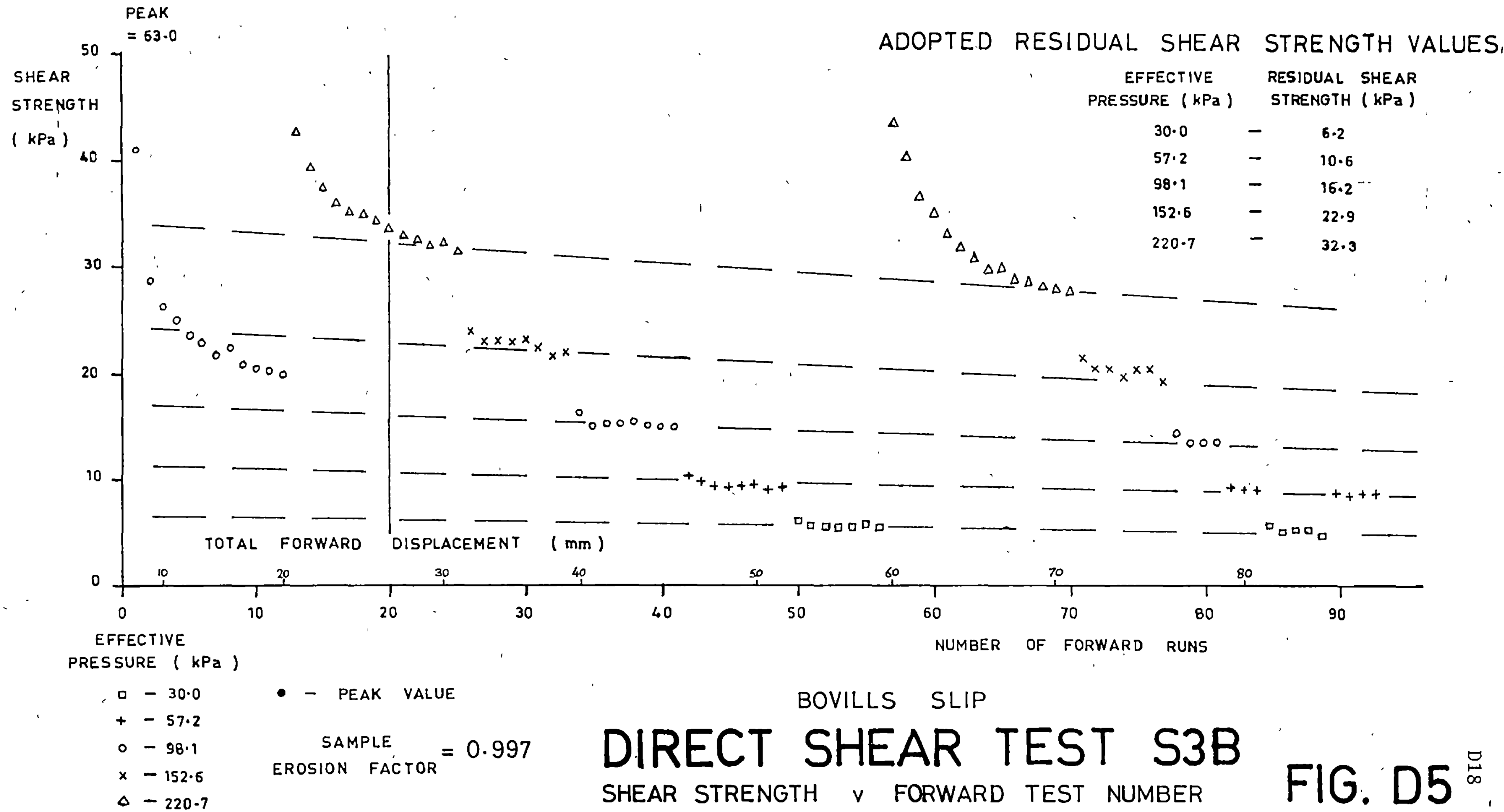
SAMPLE EROSION FACTOR = 0·993

BOVILLS SLIP

DIRECT SHEAR TEST S3A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D4



SAMPLE EROSION FACTOR = 0·997

BOVILLS SLIP

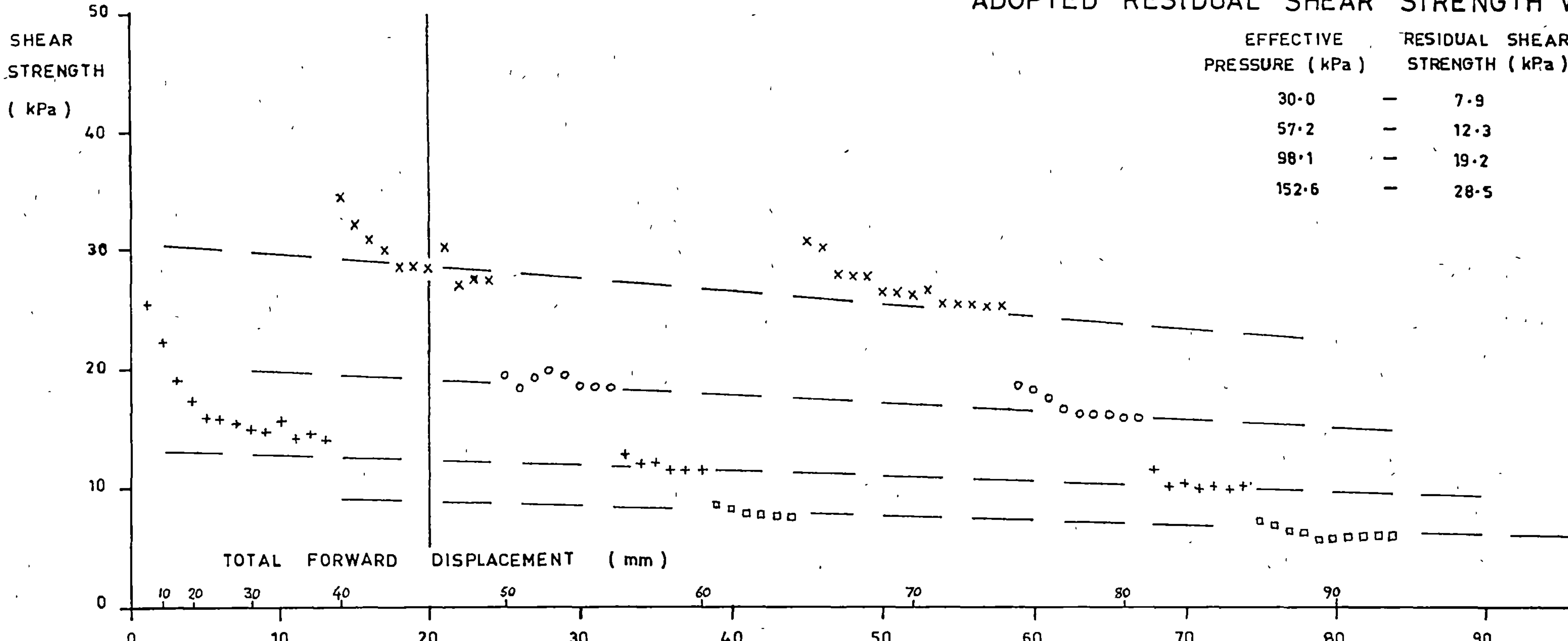
DIRECT SHEAR TEST S3B

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D5

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	7·9
57·2	—	12·3
98·1	—	19·2
152·6	—	28·5



EFFECTIVE PRESSURE (kPa)

□ — 30·0
+ — 57·2
o — 98·1
x — 152·6

• — PEAK VALUE

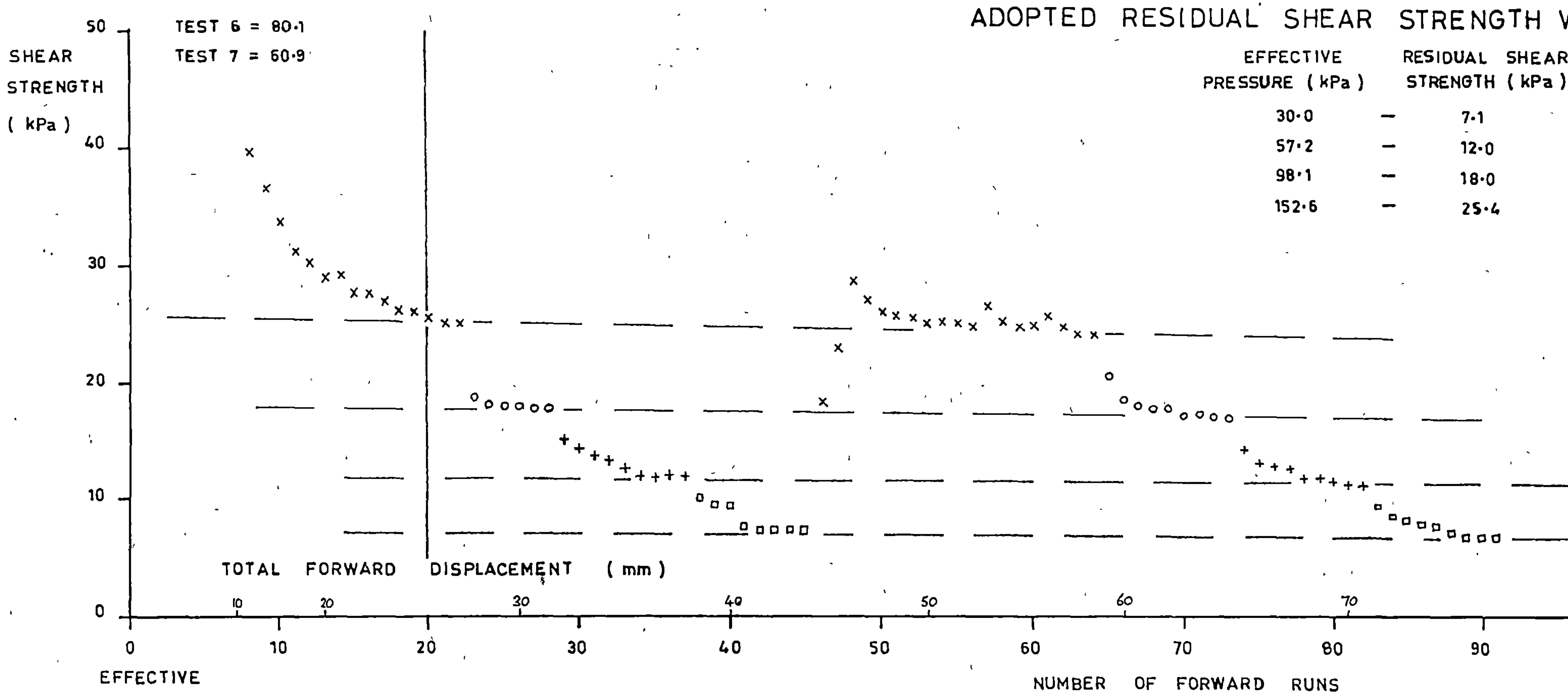
SAMPLE EROSION FACTOR = 0·996

BOVILLS SLIP

DIRECT SHEAR TEST S3C

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D6



ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	7·1
57·2	—	12·0
98·1	—	18·0
152·6	—	25·4

EFFECTIVE PRESSURE (kPa)

□ — 30·0
+ — 57·2
○ — 98·1
× — 152·6

● — PEAK VALUE

SAMPLE EROSION FACTOR = 0·999

BOVILLS SLIP

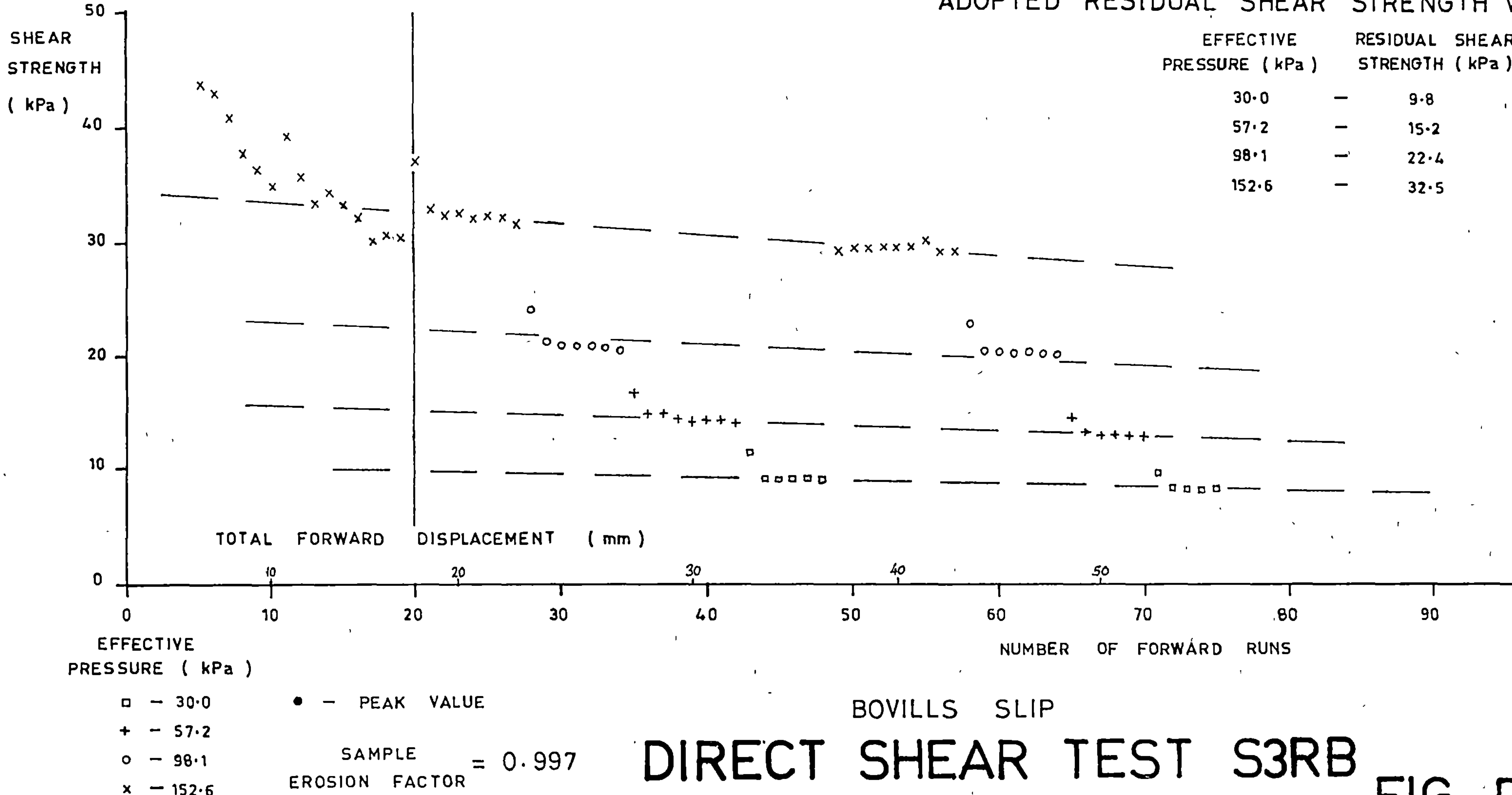
DIRECT SHEAR TEST S3RA

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D7

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	9·8
57·2	—	15·2
98·1	—	22·4
152·6	—	32·5



BOVILLS SLIP

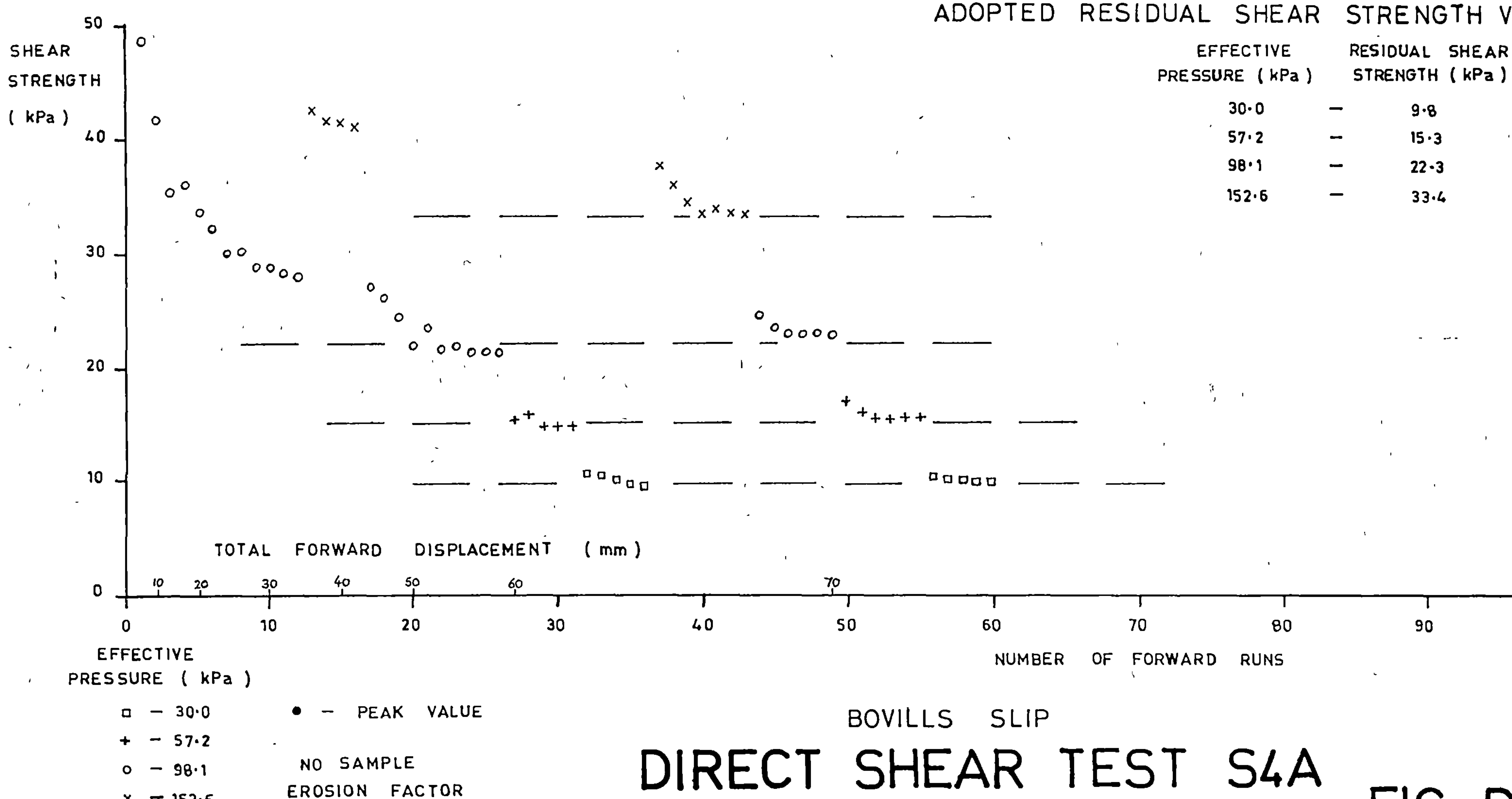
DIRECT SHEAR TEST S3RB

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D8

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	9·8
57·2	—	15·3
98·1	—	22·3
152·6	—	33·4



BOVILLS SLIP

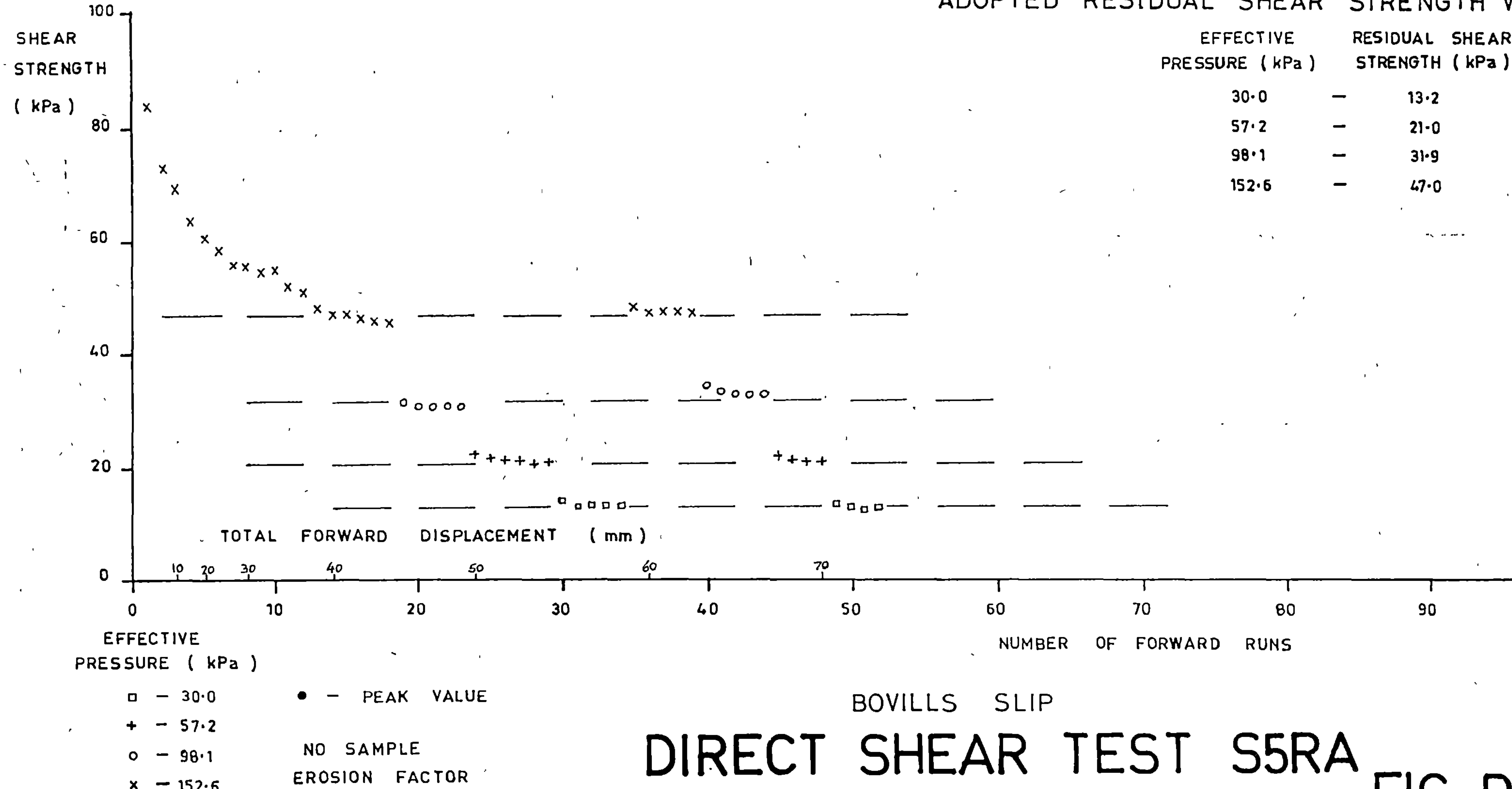
DIRECT SHEAR TEST S4A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D9

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	13·2
57·2	—	21·0
98·1	—	31·9
152·6	—	47·0

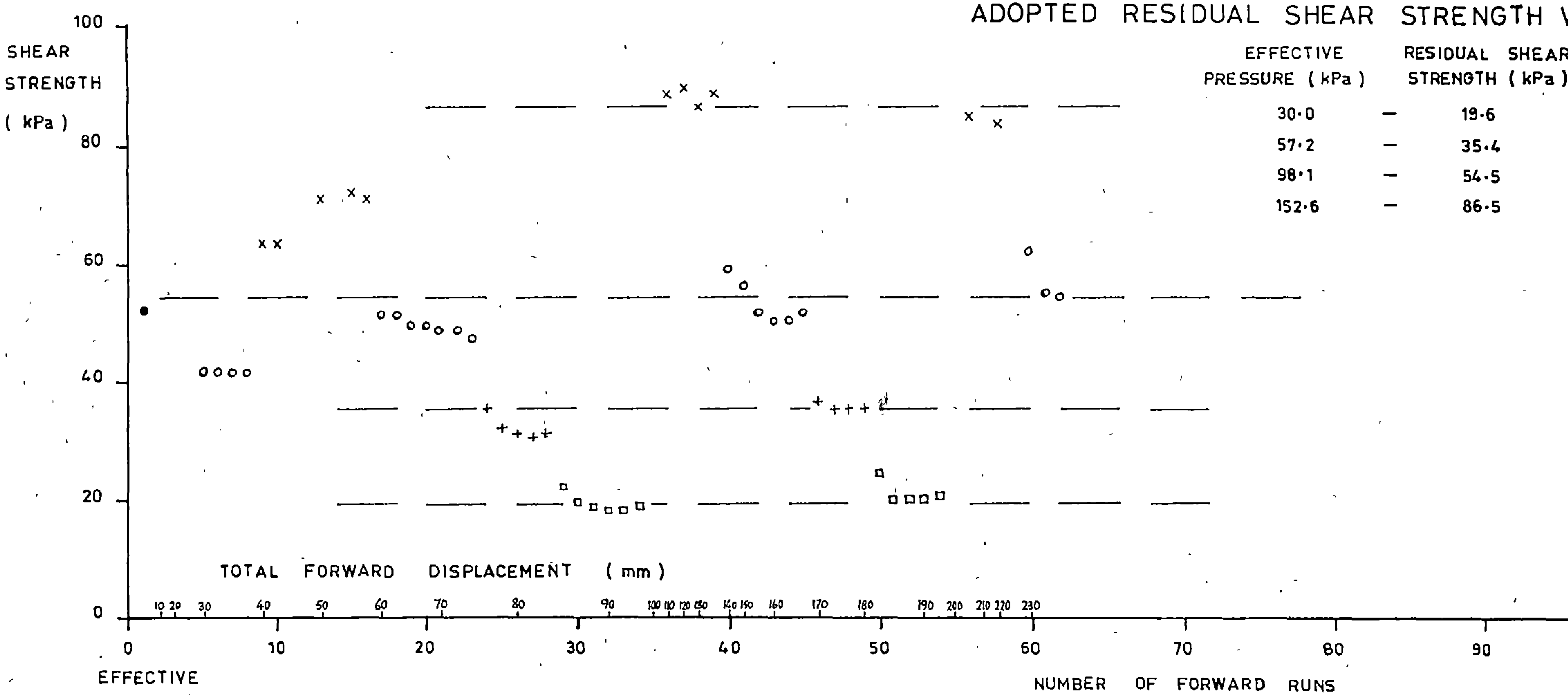


BOVILLS SLIP

DIRECT SHEAR TEST S5RA

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D10



ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	19·6
57·2	—	35·4
98·1	—	54·5
152·6	—	86·5

EFFECTIVE PRESSURE (kPa)

□ — 30·0
\+ — 57·2
o — 98·1
x — 152·6

● — PEAK VALUE

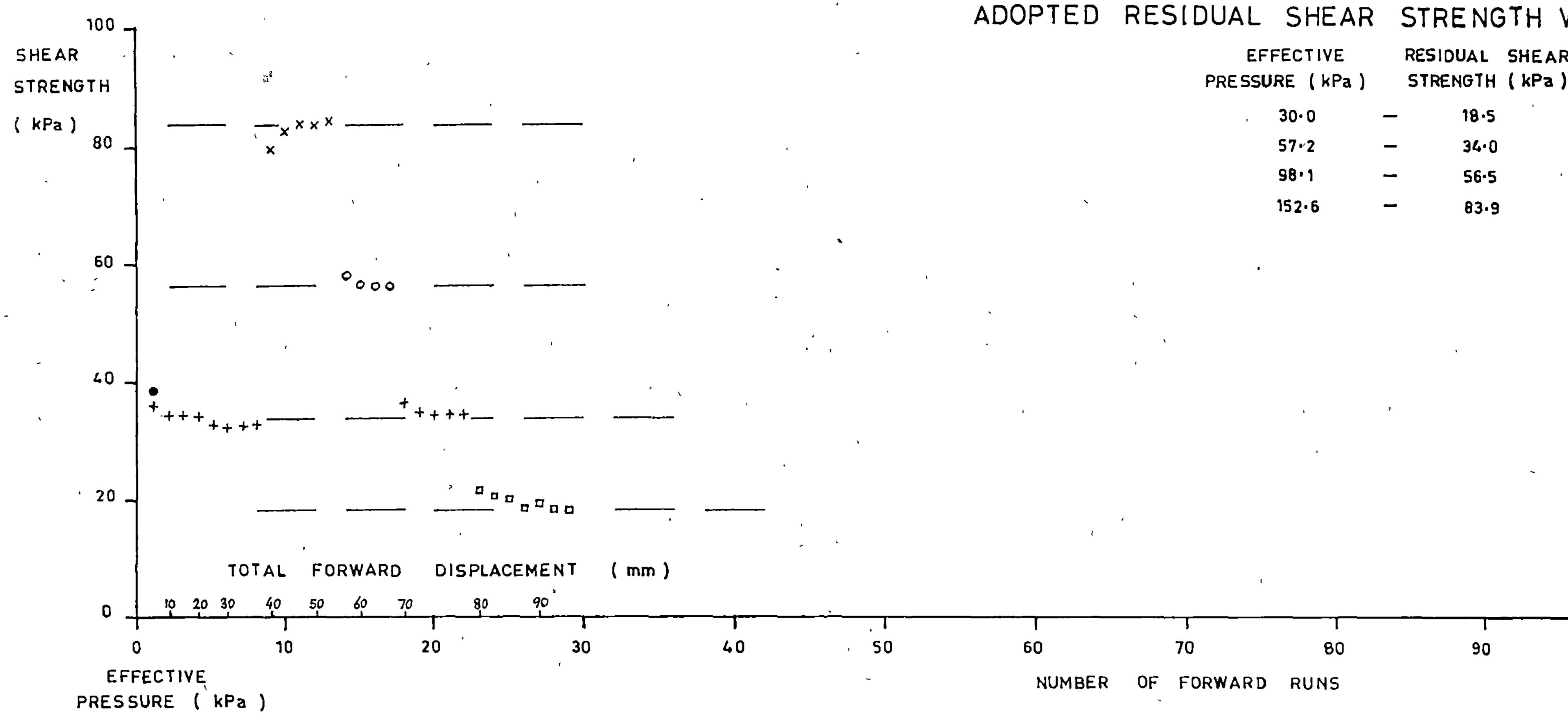
NO SAMPLE EROSION FACTOR

BOVILLS SLIP

DIRECT SHEAR TEST S9A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D11



ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	18·5
57·2	—	34·0
98·1	—	56·5
152·6	—	83·9

□ — 30·0
+ — 57·2
o — 98·1
x — 152·6

● — PEAK VALUE

NO SAMPLE EROSION FACTOR

BOVILLS SLIP

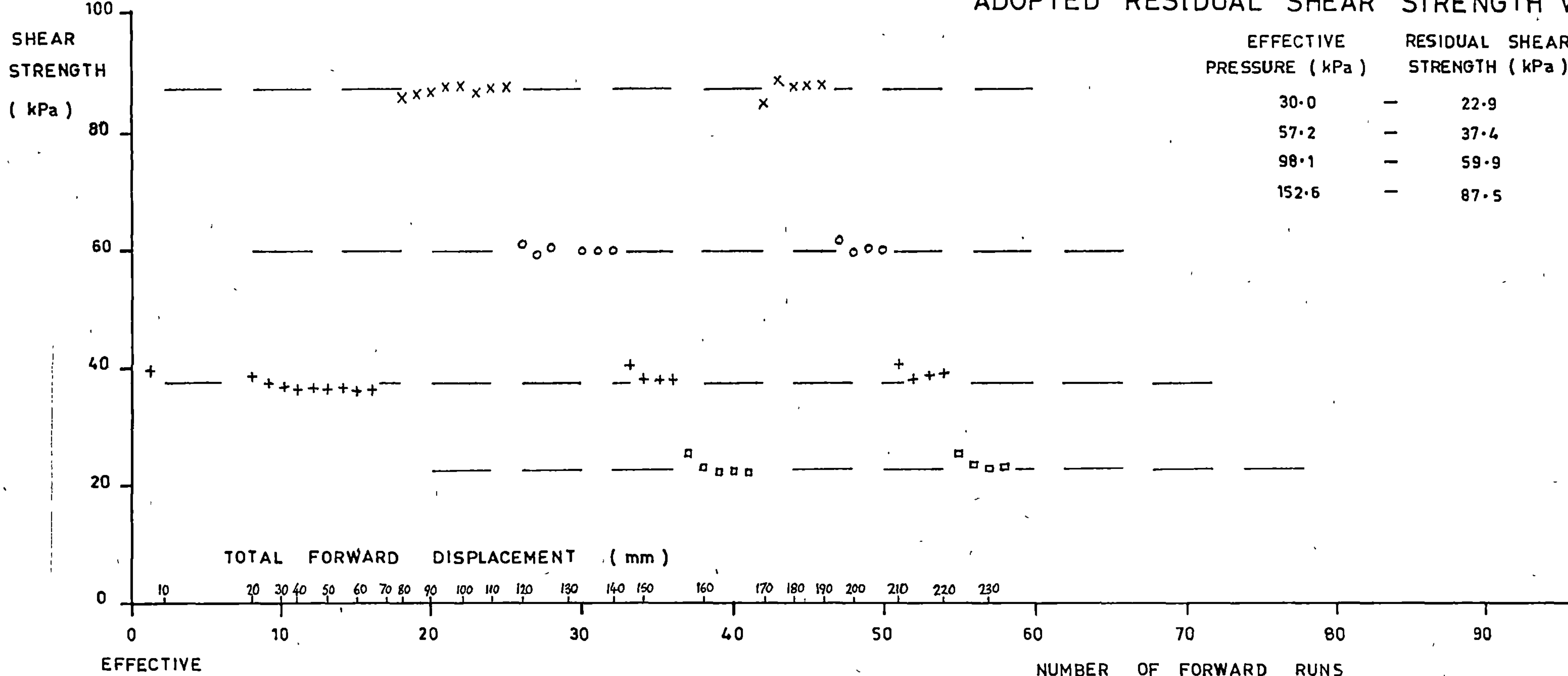
DIRECT SHEAR TEST S10A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D12

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	22·9
57·2	—	37·4
98·1	—	59·9
152·6	—	87·5



EFFECTIVE PRESSURE (kPa)

□ — 30·0
\+ — 57·2
o — 98·1
× — 152·6

● — PEAK VALUE

NO SAMPLE EROSION FACTOR

BOVILLS SLIP

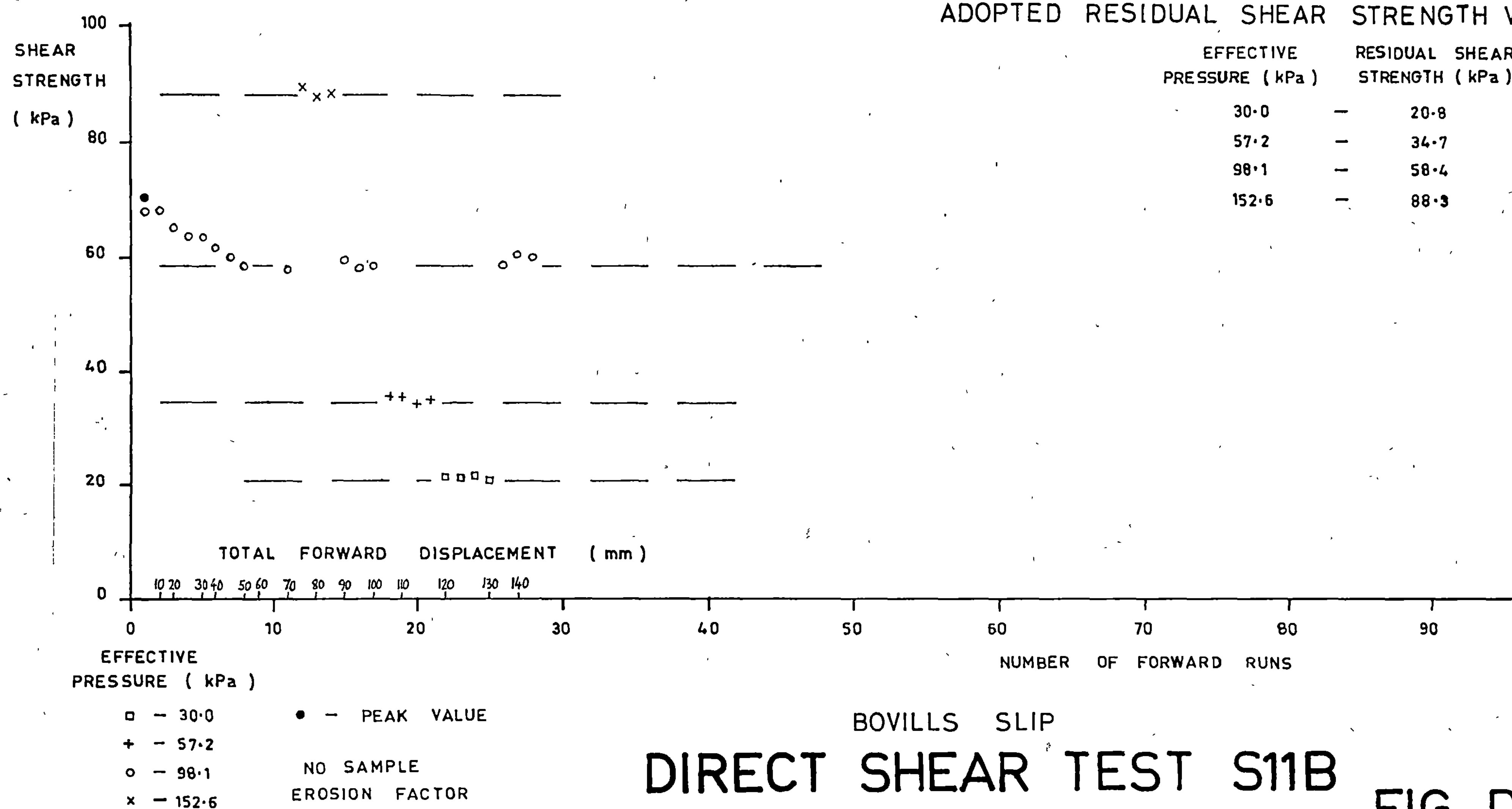
DIRECT SHEAR TEST S11A

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D13

ADOPTED RESIDUAL SHEAR STRENGTH VALUES

EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	—	20·8
57·2	—	34·7
98·1	—	58·4
152·6	—	88·3



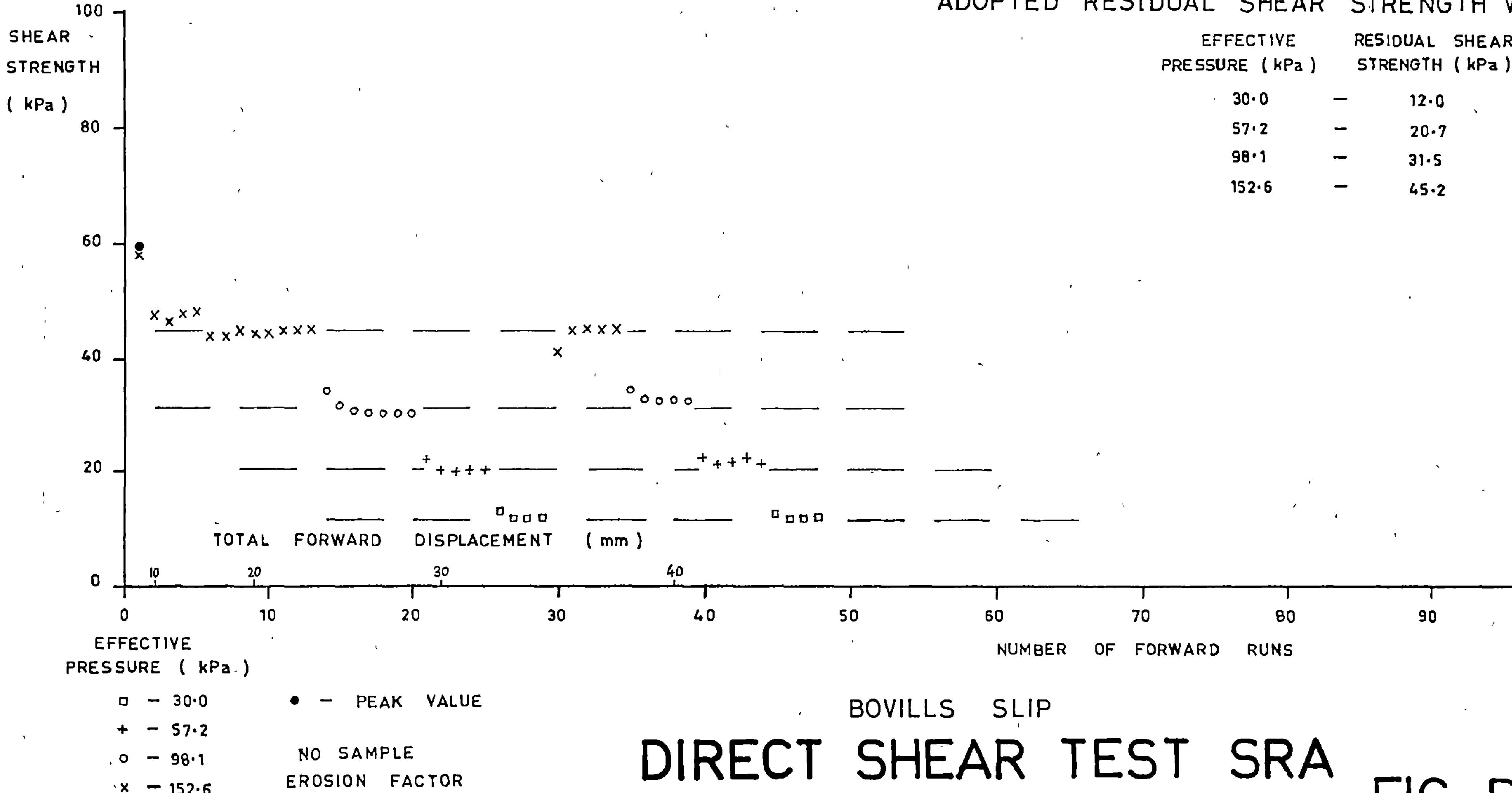
NO SAMPLE EROSION FACTOR

BOVILLS SLIP

DIRECT SHEAR TEST S11B

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D14



ADOPTED RESIDUAL SHEAR STRENGTH VALUES

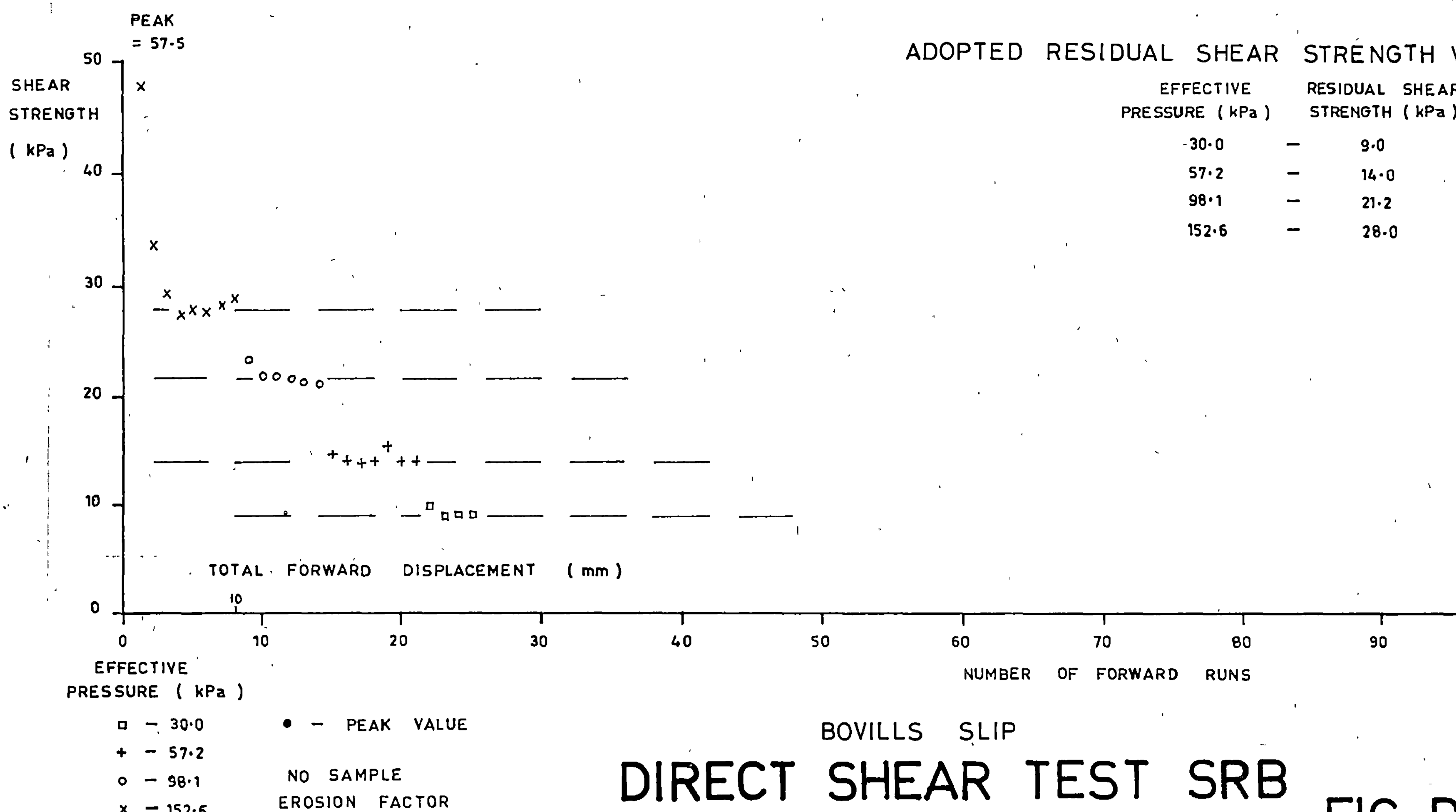
EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	–	12·0
57·2	–	20·7
98·1	–	31·5
152·6	–	45·2

BOVILLS SLIP

DIRECT SHEAR TEST SRA

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D15



ADOPTED RESIDUAL SHEAR STRENGTH VALUES

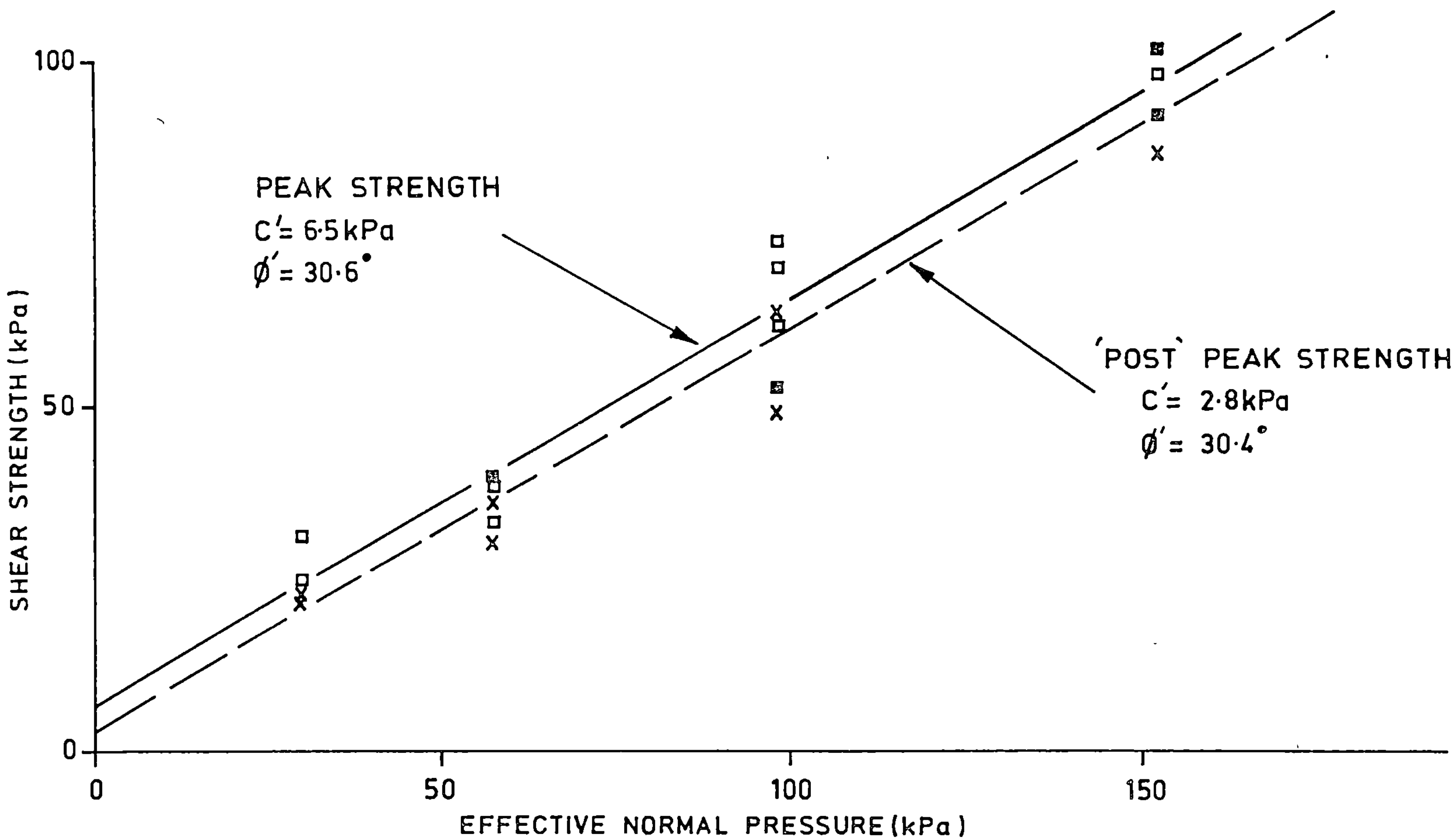
EFFECTIVE PRESSURE (kPa)		RESIDUAL SHEAR STRENGTH (kPa)
30·0	–	9·0
57·2	–	14·0
98·1	–	21·2
152·6	–	28·0

BOVILLS SLIP

DIRECT SHEAR TEST SRB

SHEAR STRENGTH v FORWARD TEST NUMBER

FIG. D16



LEGEND

□ PEAK STRENGTH

x POST PEAK STRENGTH (AT 7mm BOX DISPLACEMENT)

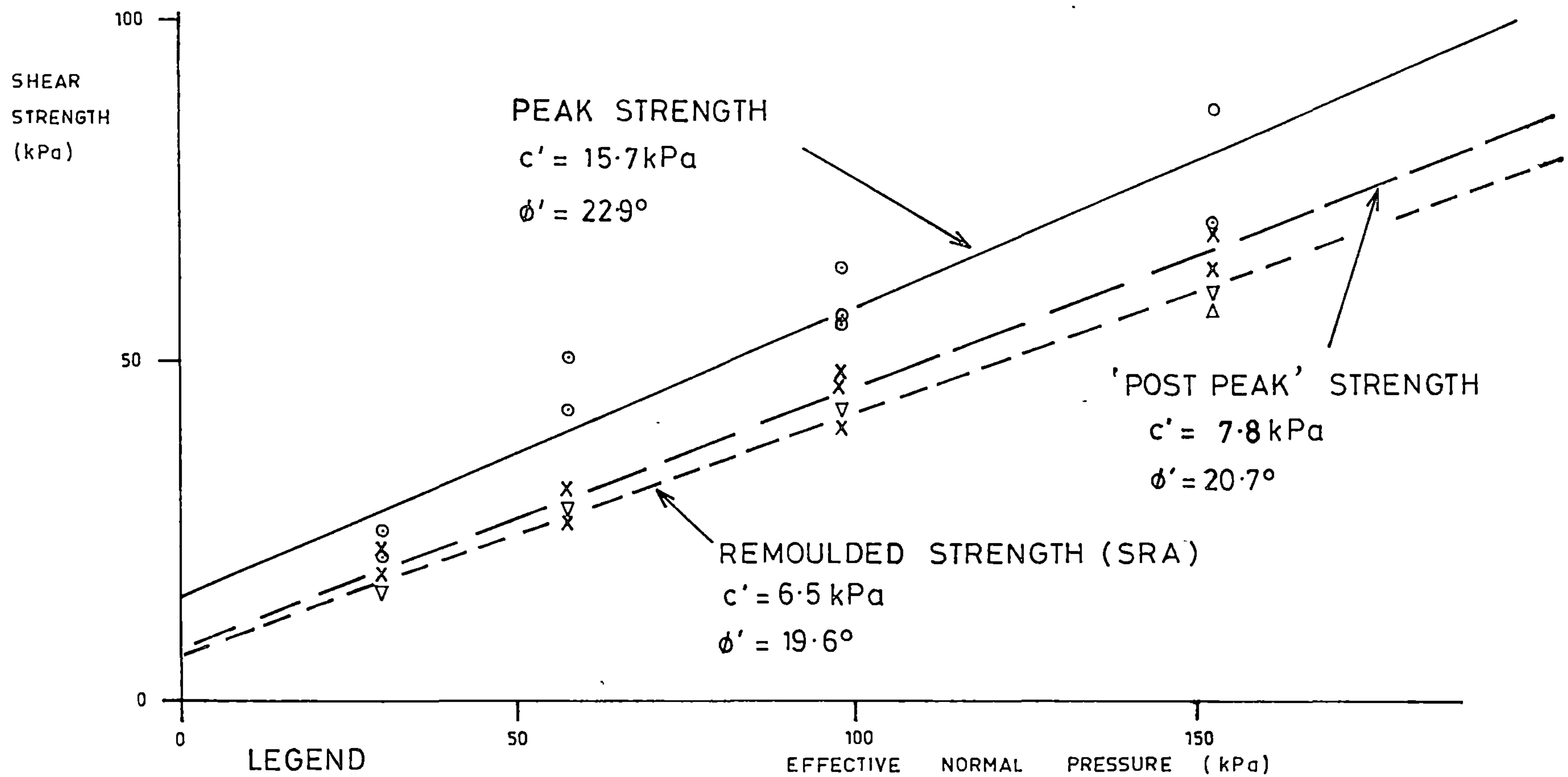
DETAILED RESULTS IN TABLE D3

BOVILLS SLIP

DIRECT SHEAR TESTS

STRENGTH OF LOWER PLASTICITY SAMPLES

FIG. D17



LEGEND

⊙ PEAK STRENGTH

X 'POST PEAK' STRENGTH (AT 7mm BOX DISPLACEMENT)

∇ SRA } PEAK STRENGTH FOR REMOULDED SAMPLES
Δ SRB

DETAILED RESULTS IN TABLE D4

BOVILLS SLIP

DIRECT SHEAR TESTS

STRENGTH OF HIGHER PLASTICITY SAMPLES

FIG. D18

APPENDIX E

TRIAXIAL TESTS

E.1 INTRODUCTION

For the analysis of first time failures the most appropriate laboratory parameters are those for the fully softened condition (Chapter 5). Triaxial tests were carried out in order to determine the fully softened strength parameters. This appendix includes a description of test apparatus used, an account of test procedures, and presentation of the results. The relationship of these results to other soil parameters is discussed in Chapter 5. Tables and figures are included at the end of this Appendix.

E.2 APPARATUS

A standard triaxial cell has been used for all the tests reported here. Strain controlled tests have been conducted with load application by a motorised loading frame. A force transducer allowed the load to be monitored by digital readout and chart recorder. Strain was measured by a dial gauge and a transducer. As the rate of loading was constant it was not necessary to use the transducer. Regular readings of the dial gauge allowed the strain to be calculated at any particular time. The rate of loading was controlled by a system of gears.

Cell pressure and pore pressure were controlled by separate constant pressure mercury pot systems. Pressures were measured by transducers and monitored by digital readout and chart recorder.

Volume change observations during consolidation stages or drained tests could be carried out by a transducer controlled volume measuring device. Volume changes (i.e. water quantities passing through the device, into or out of the sample) were monitored by digital readout and chart recorder.

E.3 TEST PROCEDURES

Of the twenty-two 38 mm diameter undisturbed samples of silty clay colluvium obtained during the field investigation only eight were suitable for triaxial testing after extrusion in the laboratory. Samples of lower plasticity soil (which fail by turbulent shear, see Chapter 5) were particularly difficult to extrude because of the high friction angle of the overconsolidated soil. The eight samples tested are identified in Table E.1.

Seven consolidated undrained tests with pore pressure measurements and one fully drained test were carried out. The undrained tests were preferred to the drained tests as they provided information on pore pressure changes and therefore more information on the failure envelope. Of the seven undrained tests, four were staged with tests conducted at four different cell pressures for the one sample. The advantages and disadvantages of staged tests are discussed later. The cell pressures for all of the tests were chosen in order to obtain strength parameters in the stress range consistent with overburden pressure. Filter paper drains were used in all of the tests.

All of the samples were collected in summer when conditions were dry and most were not fully saturated when loaded into the triaxial cell. Landslip failures occur in winter when the soils are likely to be fully saturated. In order to obtain parameters at fully saturated conditions a back pressure of 40 kPa was applied to all of the samples before testing. A back pressure of 40 kPa represents the maximum pore water pressure for soil in the failure zone at Bovills Slip. The degree of saturation was estimated before and after application of the back pressure by checking pore pressure parameter B (Table E.2).

The length of each test was controlled by the rate of loading. The rates used are shown in Table E.2. They represent strain rates in the range 0.003% per minute to 0.009% per minute.

E.4 TEST RESULTS

E.4.1 Introduction

Full records of the pre-test saturation, consolidation, and loading results are available in files and on chart records in the Department of Mines library. Calculation sheets for each test are also available. The results of each test are presented here in Figures E1 to E5 in the form of p-q stress path diagrams (Lambe and Whitman, 1969). Strain is also shown on the diagrams. Other data on the tests and samples are given in Table E.2. The results of the tests are summarised in Table E.3 and Figures E6 and E7.

In this section some details of the interpretation and calculation of the results are discussed.

E.4.2 Failure criteria

The purpose of a failure criterion is to express the relationship between the principal stresses when the soil is in limiting equilibrium. Several failure criteria were reviewed by Bishop (1966). He concluded that the Mohr-Coulomb criterion was the only simple criterion of reasonable generality. The criterion may be written:

$$\zeta = C' + \sigma' \tan \phi$$

where ζ = shear strength across rupture plane

C' = effective cohesion

σ' = effective normal stress across rupture plane

ϕ = angle of internal friction

The Mohr-Coulomb failure criterion has been used in the analysis of shear strength results for this project.

In determining the values of C′ and ϕ′ it is necessary to decide at which point during the test actual *failure* occurs. In some tests brittle failure occurs and distinct shear planes develop. In other tests, plastic barelling of the sample occurs in which case the maximum shear strength or shear strength at 20% strain may be used. In the samples tested here a combination of barelling and brittle failure occurred.

In the case of the drained test (T3) failure was defined as the maximum deviator stress, $(\sigma_1'-\sigma_3')$ max, which occurred at 18.5% strain. For the consolidated undrained tests two definitions of failure were used. The maximum ratio of principal stresses, (σ_1'/σ_3') max, occurred at a low strain, whereas the maximum deviator stress, $(\sigma_1'-\sigma_3')$ max, occurred when the strain was significantly higher (Figures E8 and E9). The stress path between the two points follows the *Coulomb line* and the sample may be regarded as being in a stabilized state of failure (Kezdi, 1980).

The two different definitions of failure will result in different values of C′ and ϕ′. Bishop and Henkel (1962) suggest that the practical significance of this difference is usually negligible whereas Leonards (1982) quotes an example where large differences in ϕ′ result. In this project the different definitions of failure result in only small differences in strength in the stress range tested (Table E.3 and Figures E7 and E8).

E.4.3 Staged tests

Four of the consolidated undrained tests were staged. In each of these tests four different cell pressures were used during the testing of each sample. Staged tests have the advantage that more information can be obtained from a single sample. The results presented here (Figures E1 to E5) show that the stress path followed the Coulomb line over a large strain (1% to about 17%). In each test the cell pressure for the final stage was chosen to allow the stress path to cover the same range as in an earlier stage. In every case the Coulomb line from the final stage closely overlapped an earlier stage. Thus the Coulomb lines from each stage could be connected to form a single straight failure envelope. It is not known whether such consistent envelopes are usual for such tests or are partly due to a fortunate cancelling of errors (errors due to deformation of the rubber membrane and changes in cross-sectional area would be greater at larger strains). However, it is clear that failure envelopes may be drawn with confidence for each of the four staged tests presented here.

The alternative to staged tests is to separately test different samples of the same soil at different cell pressures and to assume that the results will fall on a single failure envelope (see results on Figure E1). These tests involve less strain and consequently less error might be expected in calculating the results. However, the major problem with single tests is the assumption that the soils are similar to the extent that the results will fall on the same failure envelope. Comparing the staged tests on similar soils (e.g. T8 and T9, Figures E2 and E3 and Table E.3) it can be seen that although the slope of the failure envelope is consistent, the cohesion intercept can vary from test to test. Failure envelopes may be parallel without necessarily being coincident. Attempting to draw failure envelopes between points on the individual curves from different samples could give misleading slopes. This is illustrated in Table E.3 where the analysis of the combined data results in friction angles (ϕ') larger than the individual angles. In the case where failure is defined as the maximum deviator stress the analysis of the combined data gives a ϕ' greater than the ϕ' from any of the individual tests.

The results presented here suggest that staged tests have been more useful than individual tests in providing an estimate of the slope (tan ϕ') of the failure envelope. The cohesion intercept is discussed later.

Each stage in the tests was continued until the maximum deviator stress, $(\sigma_1' - \sigma_3')$ max, was reached. Bishop and Henkel (1962) suggest that each stage need only be continued until the maximum ratio of the principal stresses, (σ_1'/σ_3') max, is reached. Their approach would allow the four stages to be completed at lower strain, but the former approach provides more information on the 'Coulomb line' and allows failure envelopes to be calculated for both definitions of failure.

E.4.4 Membrane and filter drain corrections

The use of rubber membranes and filter paper drains restrains the sample during the test and introduces an error in the measured stresses. For plastic failure, when samples become barrel shaped, membrane corrections proposed by Henkel and Gilbert (1952) are sometimes applied. Bishop and Henkel (1962) discuss membrane and filter drain corrections and suggest a combined correction of about 14 kPa is appropriate for a 38 mm diameter sample. Chandler (1966) indicates the final correction may be as high as 70 kPa at large strains and Pachakis (1976) reported that allowing for corrections could reduce the value of ϕ' by up to 13%. Appropriate corrections at large strain, in samples that have failed partly by brittle failure, are clearly difficult to determine.

The effect on the failure envelope of the restraint imposed by the filter drain and the membrane may be considered in two components. It will cause an apparent increase in effective cohesion, C', and may also cause an apparent increase in friction angle, ϕ'. The actual value of C' determined from these triaxial tests is not important as it has not been used in analysis. Thus any errors in C' caused by restraint may be ignored. Errors in ϕ' are important as ϕ' from triaxial tests were used as a fully softened strength parameter in the analysis of first time slides. Fully softened ϕ' was also investigated in direct shear and a comparison of all the results is given in Table 5 (main text). It can be seen that ϕ' determined by triaxial tests is very close to that determined by direct shear. In the case of the turbulent shear results ϕ' determined from triaxial tests is only slightly higher than the residual friction angle, ϕ_r'. Thus, it appears that errors in ϕ' due to membrane and filter paper restraint are small and the test results have been reported without corrections. However, in the case of the sliding shear soil where most results are available, the fully softened ϕ' adopted for analysis is slightly lower than that determined from the triaxial tests.

E.4.5 Pore pressure

The behaviour of the pore pressure and the pore pressure parameter A during the first stage of two of the triaxial tests is shown in Figures E8 and E9. The results are typical of tests on overconsolidated cohesive soils. Other data on the pore pressure parameters are given in Table E.2.

E.4.6 Cohesion

Triaxial tests on small samples tend to overestimate cohesion (Skempton, 1977) and errors due to membrane restraint and changes in cross-sectional area have more effect on cohesion than on friction angle. For these reasons, cohesion values from the triaxial tests have not been used in analysis. The fully softened cohesion, C', has been assumed to be 3 kPa, the same as the residual cohesion value, C_r'. By definition C' could not be assumed to be less than C_r'. Fully softened cohesion is discussed in Section 5.3.

TABLE E.1. TRIAXIAL SAMPLES

Sample number	*Test pit or borehole (TP or BH)*	*Depth (m)*	*Test type* U = undrained S = staged undrained D = drained
T3	TP1	3.37 to 3.44	D
T4	TP1	3.39 to 3.46	U
T5	TP1	3.34 to 3.42	U
T6	TP1	3.40 to 3.48	U
T8	TP2	2.21 to 2.28	S
T9	TP2	2.47 to 2.55	S
T18	BH6	2.57 to 2.65	S
T19	BH7	2.55 to 2.62	S

TABLE E.2. TRIAXIAL TESTS, SAMPLE AND TEST DATA

Sample number	Moisture content (%)		Atterberg limits (%)			Initial unit weight (kN/m³)	Confining pressure (kPa)	Pore pressure parameters		
	before test	after test	liquid limit	plastic limit	plasticity index			'B' before test	'A' at failure $(\sigma_1'/\sigma_3')_{max}$	$(\sigma_1'-\sigma_3')max$
T3	40.9	55.4	-	-	-	-	70	-	-	-
T4	39.0	48.3	122	36	86	≅19.3	130	-	0.22	0.12
T5	40.6	48.9	124	40	84	17.9	100	0.99	0.30	0.15
T6	42.0	51.9	-	-	-	19.3	70	0.55, 0.74, 0.89	0.28	0.07
T8	39.5	51.2	118	41	77	18.5	70, 100, 130, 90	0.35, 0.95	0.24	0.05
T9	40.1	51.1	123	40	83	17.5	70, 100, 130, 90	0.88, 0.96	0.21	0.15
T18	40.2	50.4	108	37	71	18.2	55, 70, 85, 44.5	0.81, 0.85	0.03	-0.19
T19	27.9	34.1	52	29	23	20.7	70, 100, 130, 55	0.72, 0.96	0.12	-0.08

NOTE: Sample T6 was slightly damaged after extrusion from the sample tube.

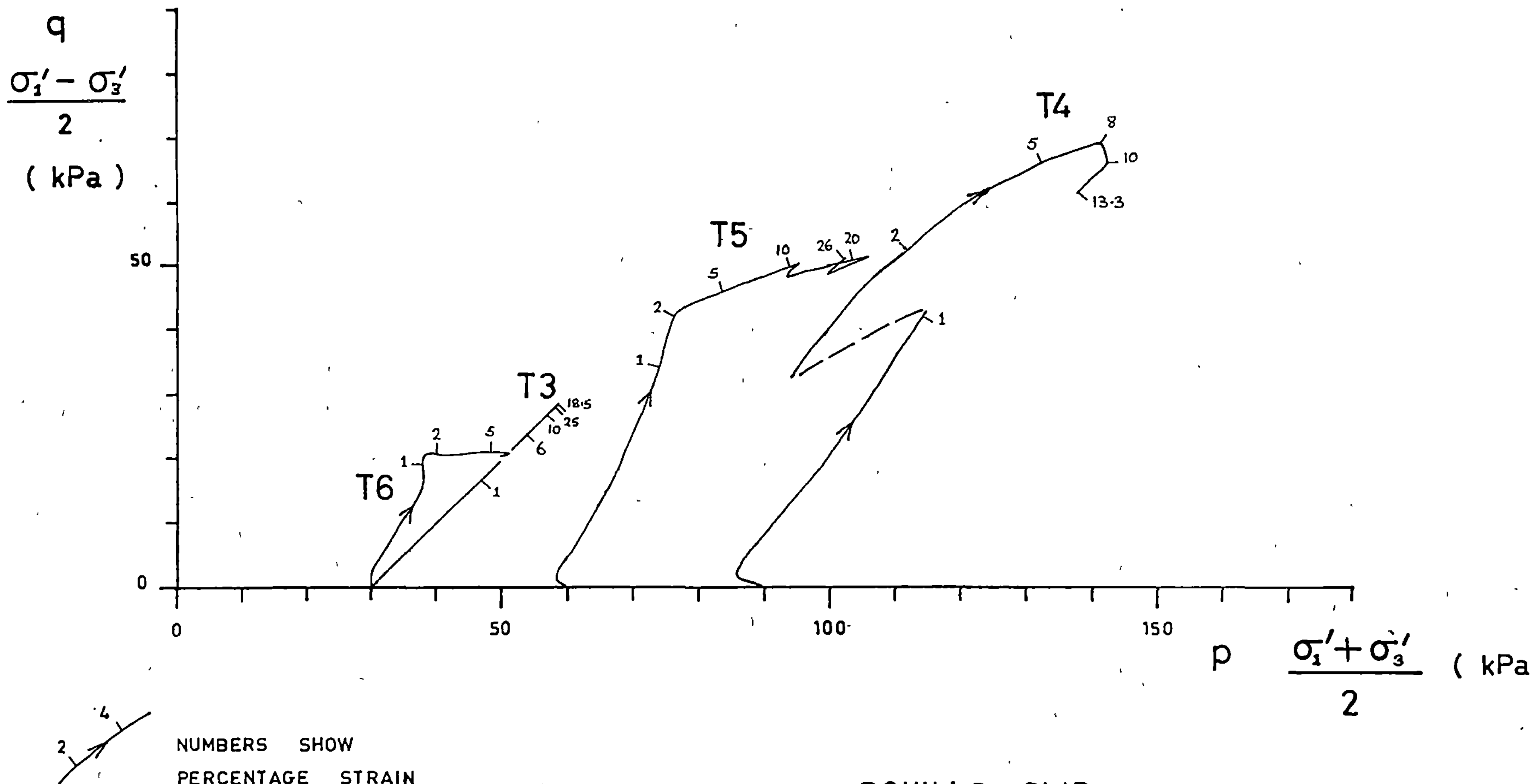
TABLE E.3. TRIAXIAL TEST RESULTS

Failure definition	Sample number	Shear stress (kPa) at failure, $p = \frac{\sigma_1 + \sigma_3}{2}$, $q = \frac{\sigma_1 - \sigma_3}{2}$						cohesion C' (kPa)	friction angle, ϕ'	R^2(%)	Comments
		p	q	p	q	p	q				
STAGED TESTS - HIGH STRENGTH (TURBULENT SHEAR)											
(σ_1'/σ_3')max	T19	63.5	44.0	148.4	88.9	205.6	117.1	71.6	49.6	14.4	30.8	99.95	
$(\sigma_1'-\sigma_3')$max	T19	113.8	72.3	181.1	105.1	233.3	127.8	83.6	56.6	20.0	28.4	99.89	
STAGED TESTS - LOW STRENGTH (SLIDING SHEAR)											
	T8	43.5	25.5	83.1	41.1	129.9	57.4	88.2	43.7	10.9	21.7	99.60)	Combined results
(σ_1'/σ_3')max	T9	46.3	23.8	74.6	34.1	123.2	50.7	83.9	38.9	8.7	20.5	99.31)	$C' = 6.4$, $\phi' = 23.1$
	T18	31.5	17.5	49.9	25.9	64.6	29.6	14.9	9.9	4.9	23.9	98.72)	$R^2 = 97.89$
	T8	59.0	32.5	97.5	46.5	141.2	61.2	108.7	49.7	12.9	20.4	99.93)	Combined results
$(\sigma_1'-\sigma_3')$max	T9	47.1	24.1	86.4	38.4	140.7	55.7	102.5	44.0	9.3	19.8	99.76)	$C' = 6.7$, $\phi' = 22.3$
	T18	49.6	25.1	62.3	28.8	74.6	31.6	25.6	14.1	6.1	21.2	97.53)	$R^2 = 97.12$

INDIVIDUAL TESTS - LOW STRENGTH (SLIDING SHEAR)

Failure definition	Sample number	p	q	Sample number	p	q	Sample number	p	q	Sample number	p	q	Comments
(σ_1'/σ_3')max	T3	58.4	28.4	T4	125.0	62.0	T5	77.9	43.4	T6	39.2	20.7	These values not
$(\sigma_1'-\sigma_3')$max	T3	58.4	28.4	T4	142.0	69.0	T5	95.1	50.1	T6	48.1	21.1	used in analysis

NOTE: R^2 is a measure of the proportion of variation in the data which is explained by the assumption that the regression equation is linear.

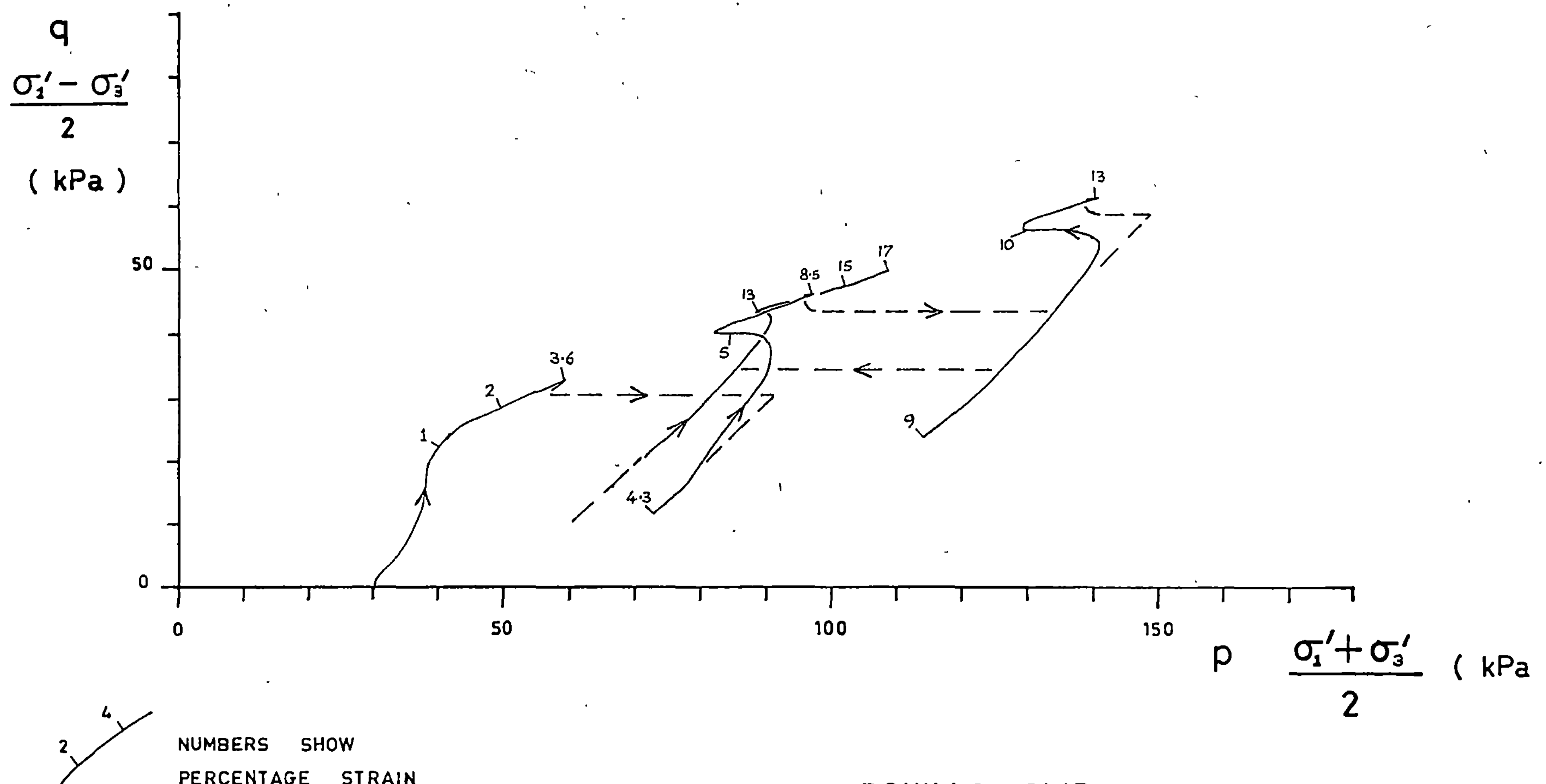


BOVILLS SLIP

TRIAXIAL TESTS T3, T4, T5, & T6

P – Q STRESS PATH DIAGRAM

FIG. E1

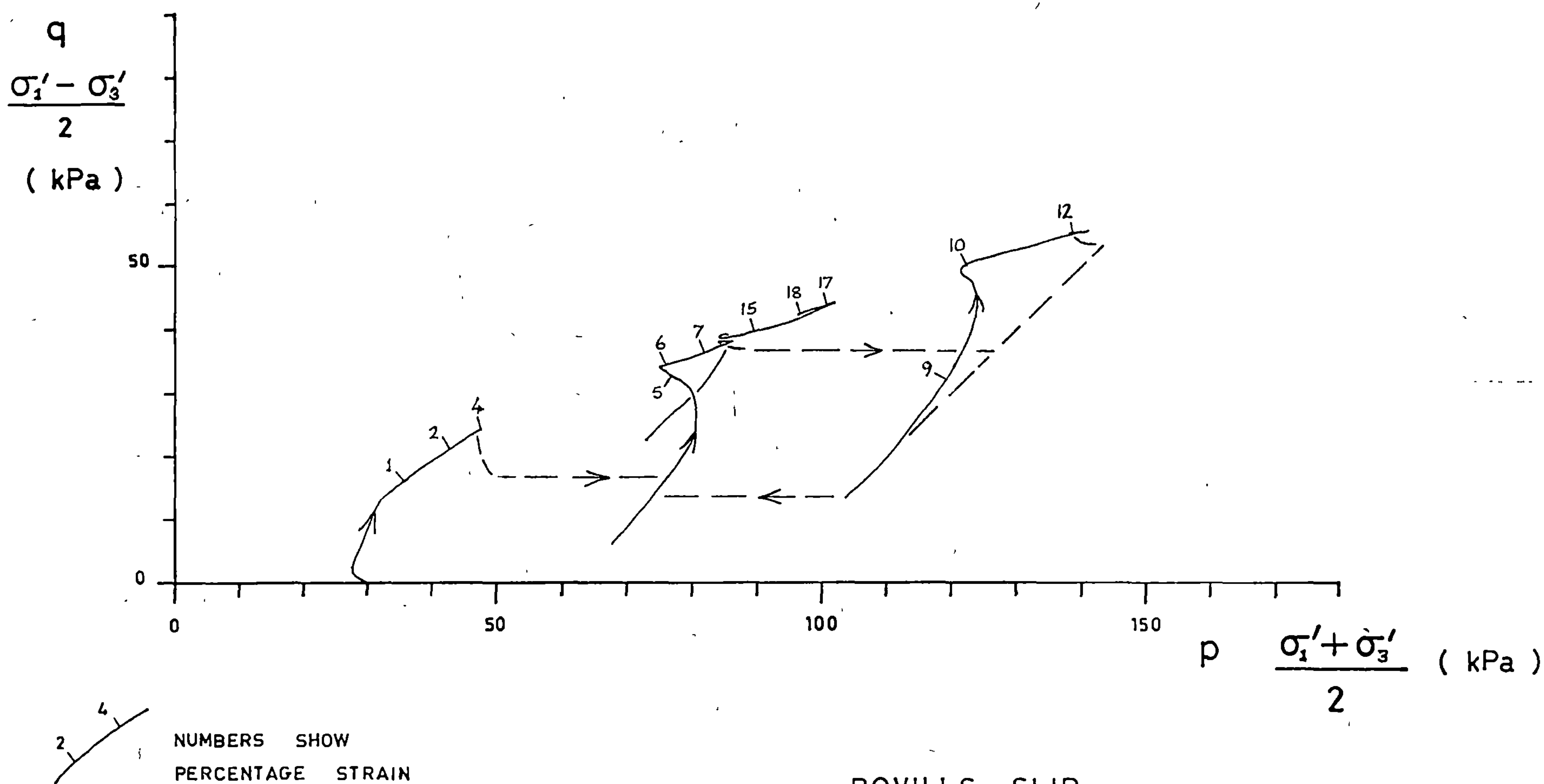


BOVILLS SLIP

TRIAXIAL TEST T8

P–Q STRESS PATH DIAGRAM

FIG. E2



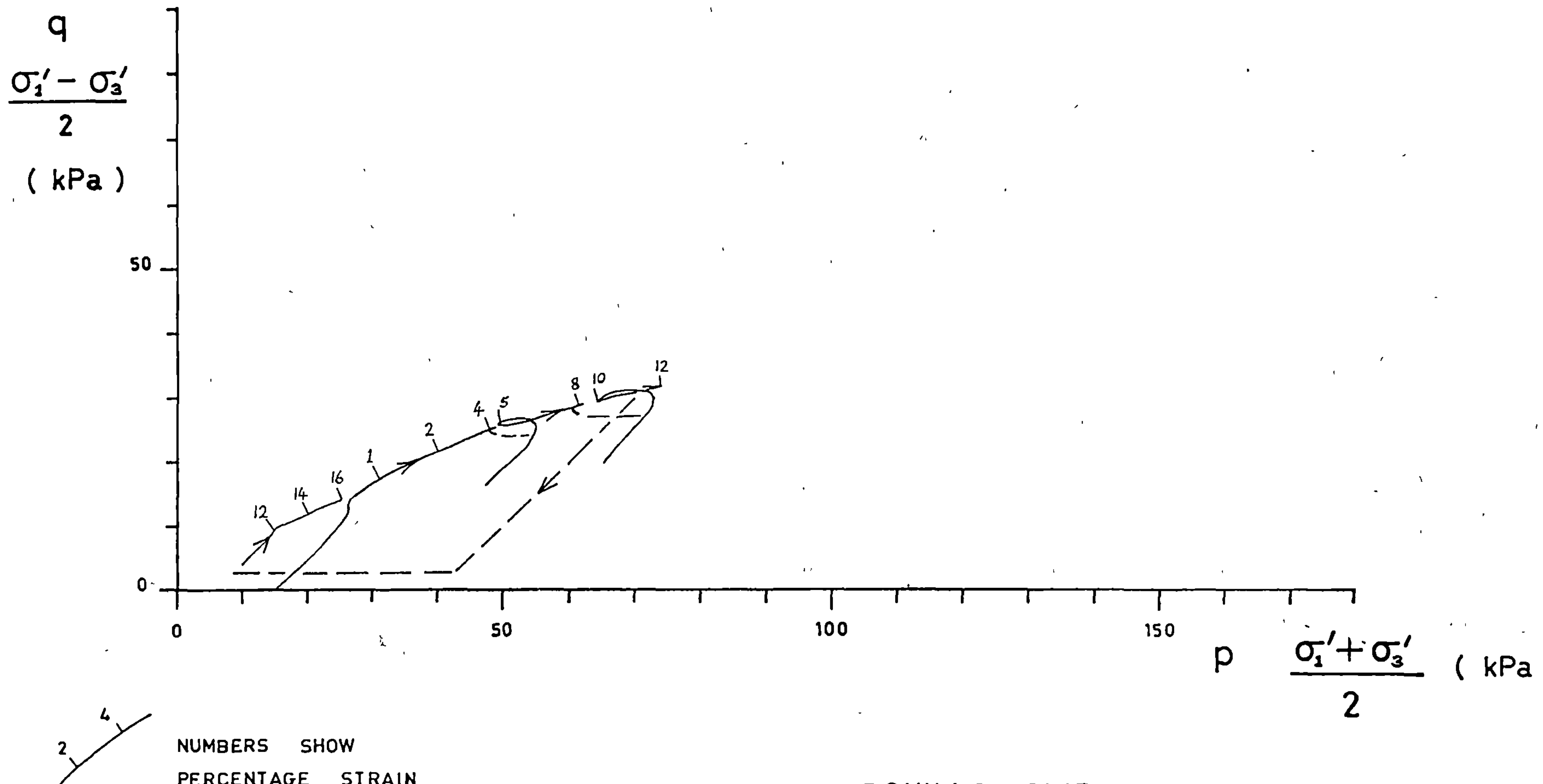
BOVILLS SLIP

TRIAXIAL TEST T9

P–Q STRESS PATH DIAGRAM

FIG. E3

E13

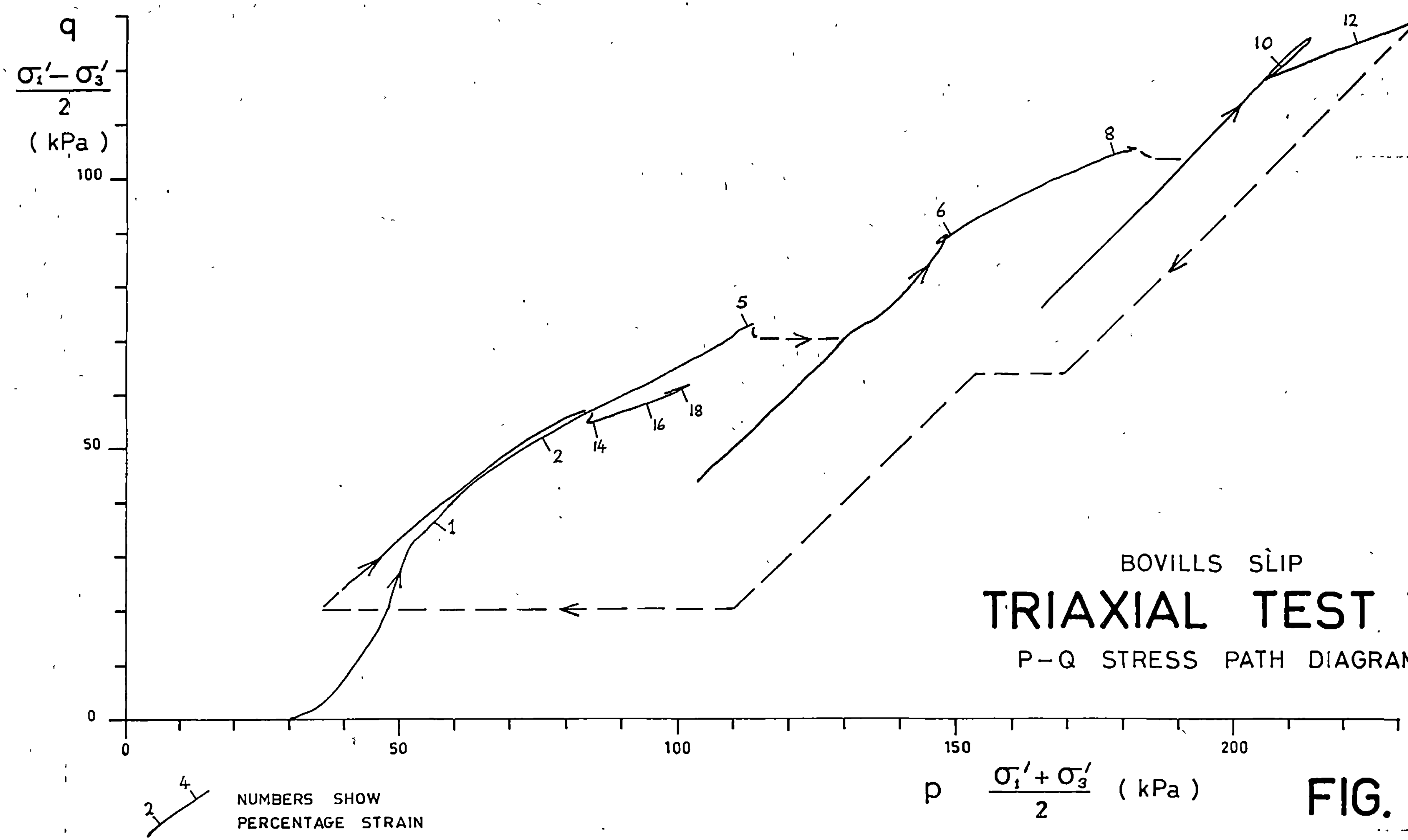


BOVILLS SLIP

TRIAXIAL TEST T18

P–Q STRESS PATH DIAGRAM

FIG. E4



BOVILLS SLIP

TRIAXIAL TEST T19

P – Q STRESS PATH DIAGRAM

FIG. E5

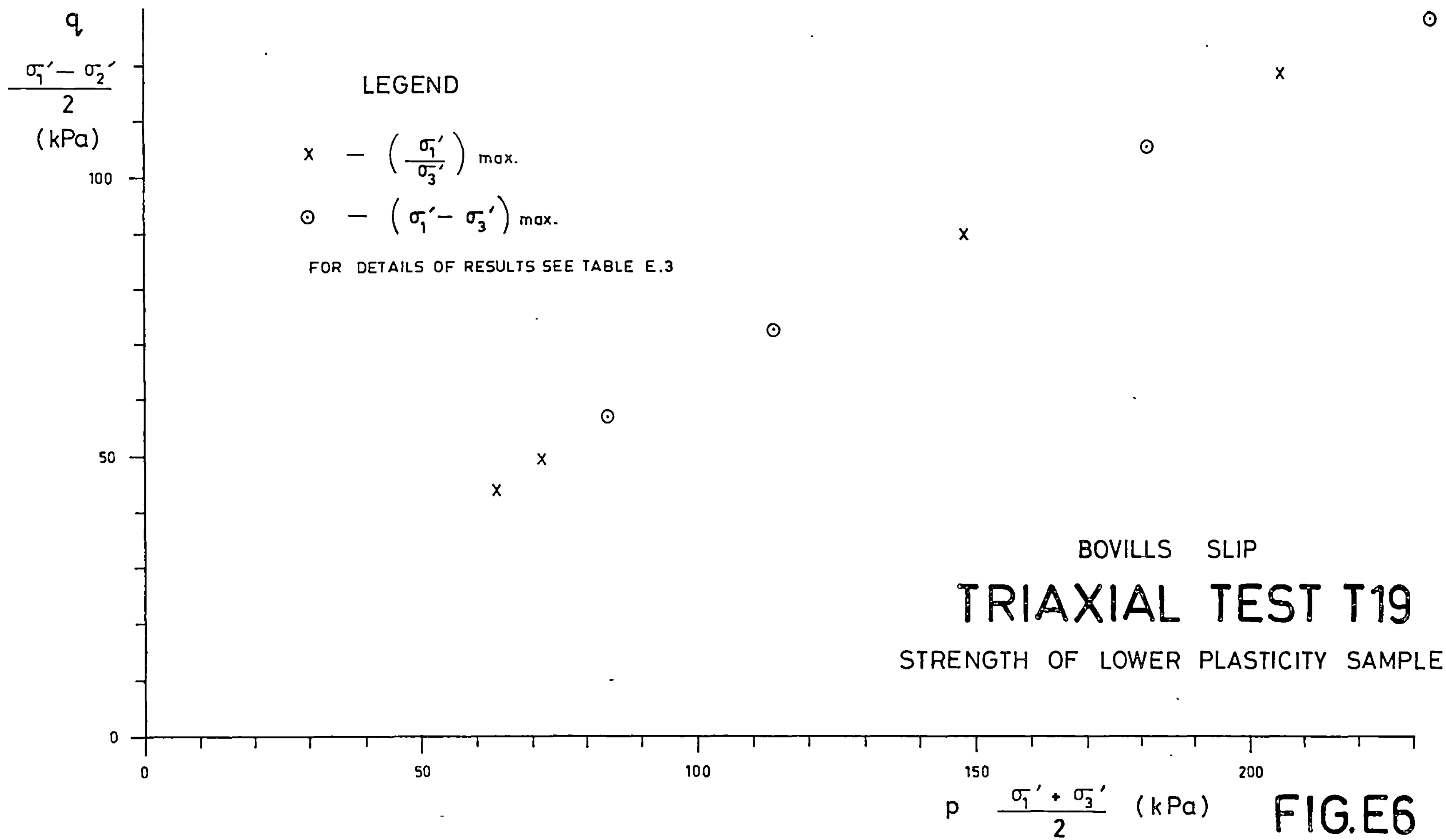
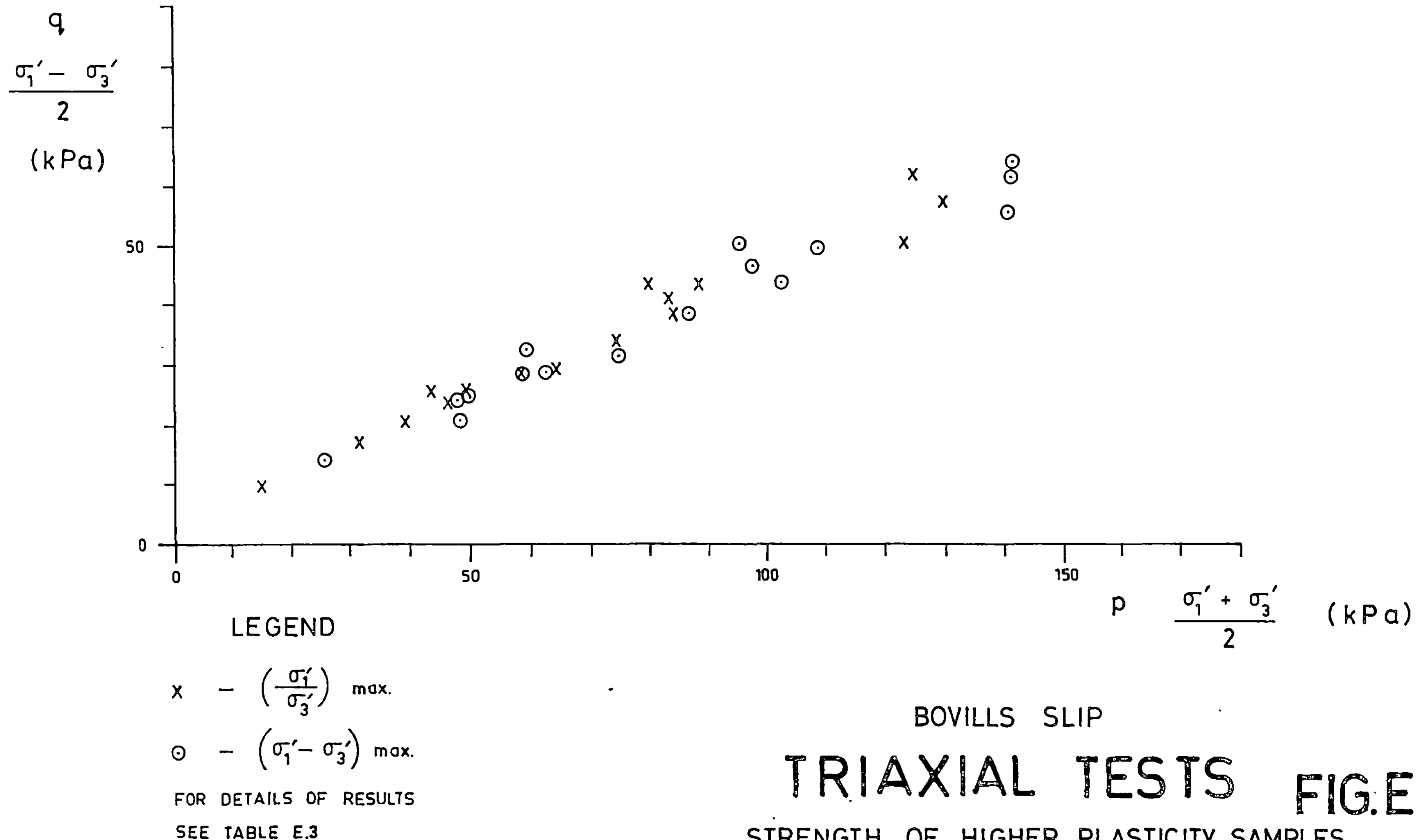

q
$\frac{\sigma_1' - \sigma_2'}{2}$
(kPa)
100
50
0
LEGEND
x — $\left(\frac{\sigma_1'}{\sigma_3'}\right)$ max.
⊙ — $(\sigma_1' - \sigma_3')$ max.
FOR DETAILS OF RESULTS SEE TABLE E.3
0
50
100
150
200
p $\frac{\sigma_1' + \sigma_3'}{2}$ (kPa)
BOVILLS SLIP
TRIAXIAL TEST T19
STRENGTH OF LOWER PLASTICITY SAMPLE

FIG.E6

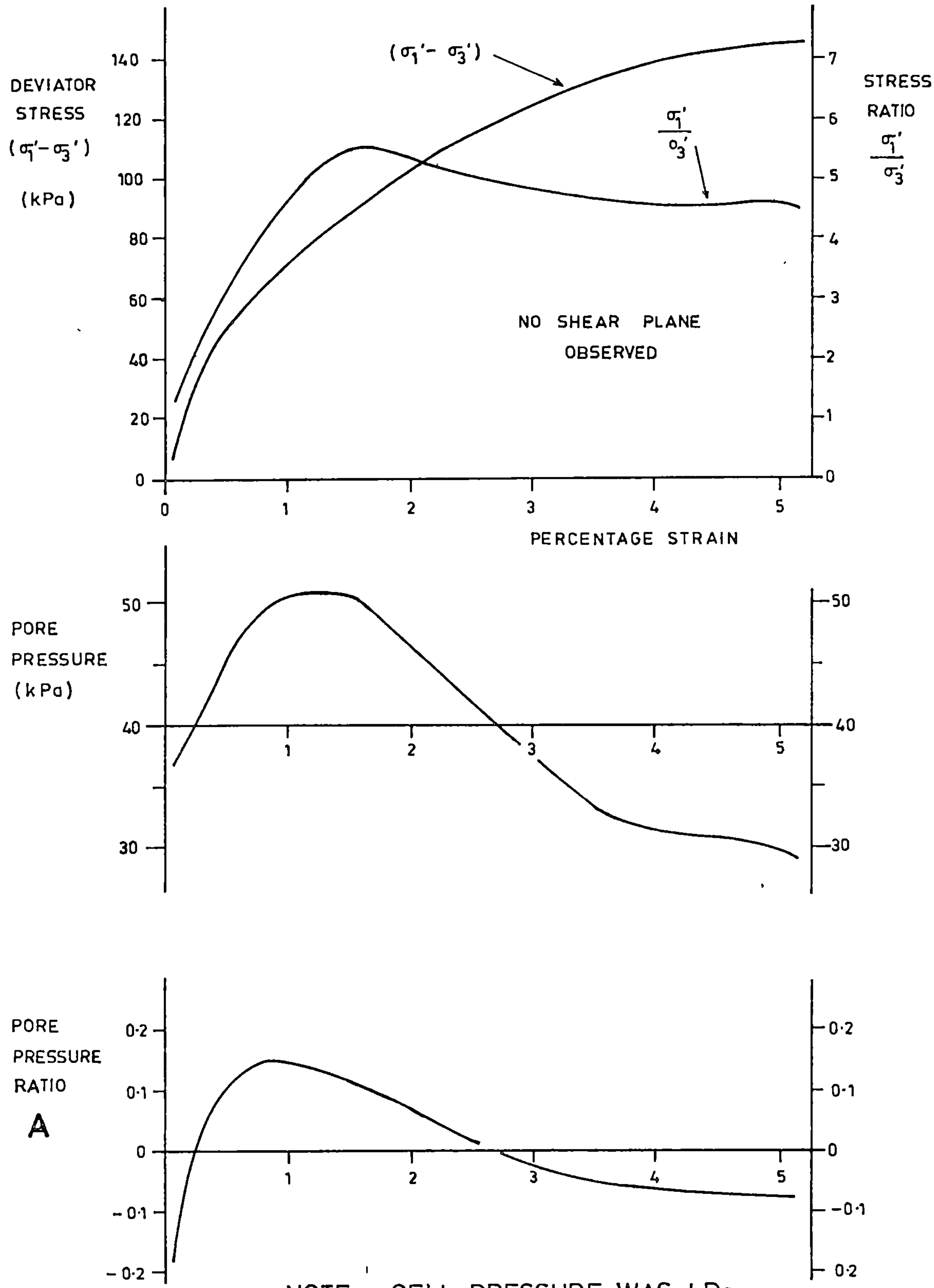


BOVILLS SLIP

TRIAXIAL TESTS

FIG. E7

STRENGTH OF HIGHER PLASTICITY SAMPLES

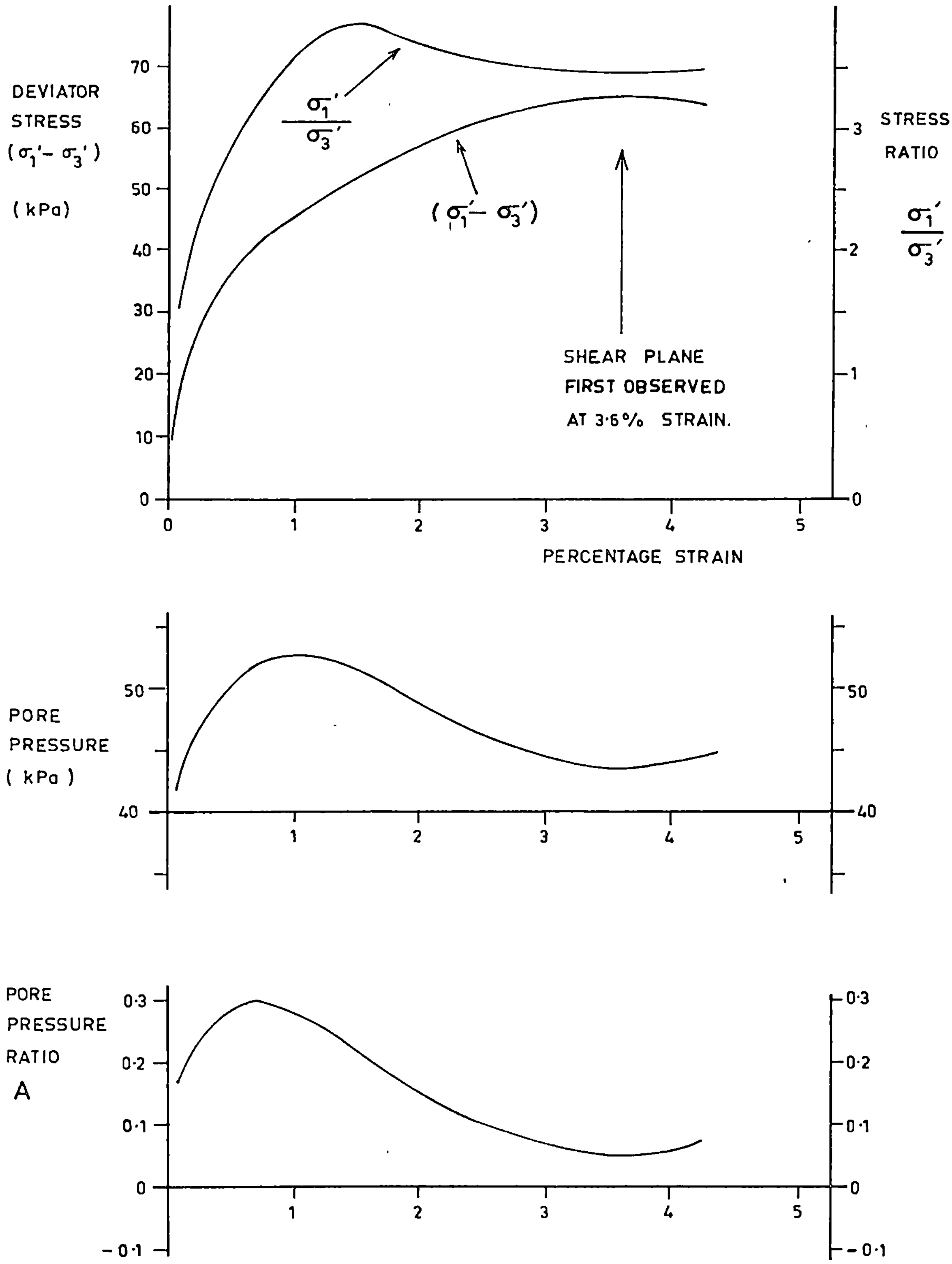


NOTE: CELL PRESSURE WAS kPa

BOVILLS SLIP

TRIAXIAL TEST T19 FIG. E8

STRESS RATIO, DEVIATOR STRESS & PORE PRESSURE



NOTE CELL PRESSURE WAS 70kPa

BOVILLS SLIP

TRIAXIAL TEST T8

FIG. E9

STRESS RATIO, DEVIATOR STRESS & PORE PRESSURE

APPENDIX F

OTHER LABORATORY TESTS

F.1 INTRODUCTION

This appendix presents and discusses the results of all the laboratory tests apart from the shear box tests (Appendix D) and the triaxial tests (Appendix E). Tables and Figures are included at the end of this Appendix.

F.2 ATTERBERG LIMITS

Atterberg limit results on the colluvial soil show a continuous variation over a wide range of plasticity (Figure F1). Tests were carried out according to Australian Standard AS 1289 (1977). All tests were carried out by the author and repeat tests on large samples at different times show that the results were reproducible. However, the reproducibility of the results by other operators in other laboratories cannot be assumed (Sherwood, 1970). The Tasmanian Department of Main Roads has carried out many Atterberg limit tests on similar basalt-derived red-brown soils. The results obtained by different operators varied, and depended to some extent on the amount of effort and time spent remoulding the soil during testing. More work on the soil led to higher values for liquid and plastic limits (R.A. Rallings, personal communication).

It was not possible to detect a consistent pattern to the variations within the colluvium. In Test pit 1 there was a higher plasticity zone between 3.3 m and 3.5 m (Figure F2). The higher plasticity soil was brown, rather than red-brown. In Test pit 2 there was a marked colour contrast at about 1.1 m. The soil above was red-brown with a plasticity index of 30 to 40%. Between 1.1 m and 2.5 m the soil was brown and yellow-brown with a plasticity index of 60 to 80% (Figure F3). Between 2.5 m and 3.1 m the plasticity was lower but there was no colour contrast.

An inspection of all of the samples from the boreholes and test pits (about 120) indicated that most of the soil was red-brown with the plasticity in the lower part of the range (plasticity index less than 50%). Higher plasticity layers and lenses were not necessarily marked by colour changes. Apart from Test pits 1 and 2, higher plasticity soil occurred in Borehole 6 between 2.5 m and 2.8 m and in Borehole 8 at 3.7 m.

Atterberg limit tests were carried out on most of the samples that were subjected to shear box and triaxial testing. The individual results are reported in Appendix D (Table D.2) and Appendix E (Table E.2). There was no evidence to suggest that the Atterberg limit results obtained after testing were different from those obtained before testing.

F.3 PARTICLE SIZE DISTRIBUTION

Full particle size distribution analyses were carried out on seven samples of silty clay colluvium (curves 1 to 7 in Figure F4). Sieve analyses were carried out on two samples of silty clay colluvium and two samples of extremely weathered basalt (curves A to D in Figure F4). The samples are identified in Table F.1. Sieve and hydrometer tests were carried out according to Australian Standard AS 1289 (1977).

Hydrometer analysis probably has similar limitations to those described for the Atterberg limit tests (i.e. the amount of work involved in sample preparation affects the results). For example, curves 6 and 7 (Figure F5) are analyses of the same sample. Analysis 6 was carried out by the author and Analysis 7 was carried out by the Hydro-Electric Commission, Tasmania. However, the results are probably reproducible for the same operator.

The clay fraction referred to in Table D.1 is an estimate of the percentage by weight of soil particles with a mean diameter of less than 2 microns. It is not necessarily equivalent to the clay content which refers to the proportion of clay minerals present irrespective of particle size. The relationship between clay content and plasticity is shown in Figure F5.

F.4 X-RAY DIFFRACTION

X-ray diffraction tests were carried out by R.N. Woolley of the Department of Mines, Tasmania. The samples were prepared by vigorously stirring about 20 g of soil in 100 ml of distilled water. The mixture was allowed to stand for five minutes after which a portion of the suspended fraction was siphoned off and allowed to dry on a glass slide. This method of sample preparation results in the exclusion of the coarse fraction of the soil and any clay particles that have not been disaggregated.

X-ray diffraction tests were carried out on samples of silty clay colluvium covering the full range of plasticity variations. Montmorillonite and kaolinite are the dominant clay minerals in all the samples tested. The proportion of montmorillonite to kaolinite increases as the total clay content increases. It appears that the content of kaolinite is fairly uniform and the plasticity variations are explained by variations in the amount of montmorillonite present in the samples. Indirect evidence of this is shown in Figure F5 which suggests that the higher plasticity soils have a greater activity index and therefore are likely to have a higher proportion of montmorillonite.

F.5 SOIL PARTICLE DENSITY

Two samples of silty clay colluvium were tested for soil particle density according to Australian Standard AS 1289 (1977). The first sample (Test pit 1, 3.0 m) was a lower plasticity soil (plasticity index of 25%) and had a soil particle density of 2.93 g/cm^3. The second sample (Test pit 1, 3.3 to 3.4 m) had a plasticity in the middle of the range (plasticity index about 50%). The soil particle density was 2.88 g/cm^3. The soil particle density of higher plasticity soils might be expected to be slightly lower.

The average soil particle density is usually assumed to be about 2.65 g/cm^3. The higher figure obtained for the soils studied is probably due to the presence of iron oxides.

Fragments of fresh or weathered basalt occur within the silty clay colluvium. The density of three fragments of fresh basalt was determined by measuring the volume of water displaced by a saturated sample and the weight in air. The average rock fragment density was 2.89 g/cm^3 with a range from 2.87 g/cm^3 to 2.90 g/cm^3.

F.6 BULK DENSITY AND DRY DENSITY

Ten determinations of the field bulk density and the dry density of the silty clay colluvium were carried out using the core cutter method (Australian Standard AS 1289, 1977). The results of the tests are given in Table F.2. The first five samples were taken in summer (March 1980) and some of them may not have been fully saturated. The other samples were taken in winter and, although close to the surface, probably were fully saturated. For this reason Samples 6 to 10 are assumed to be more representative of the winter bulk density. Bulk densities were also determined for some of the samples used for triaxial tests. The results are given in Appendix E, Table E.2.

Samples for density determinations were taken to avoid the larger rock fragments. When estimating the winter bulk density for stability analysis the presence of these rock fragments should be considered. For the purpose of analysis the bulk density of the colluvium is assumed to be about 2.04 t/m^3 and a range of 1.94 to 2.14 t/m^3 would be expected to include the 95% confidence limits. This is equivalent to a mean unit weight of 20 kN/m^3 and a range of 19 to 21 kN/m^3.

F.7 CONSOLIDATION TESTS

Consolidation tests have been carried out on two undisturbed samples of silty clay colluvium. The tests were carried out in a standard Casagrande oedometer and results have been calculated by Taylor's method (Lambe and Whitman, 1969). A summary of the test results is given in Table F.3 and Figures F6 to F8.

The results indicate that the soils are overconsolidated. The pre-consolidation pressure appears to be about 200 kPa giving an over-consolidation ratio of 4 to 8 (depending on the piezometric surface). The soils are likely to have been overconsolidated by dessication rather than by previously higher overburden pressure.

F.8 SOIL SUCTION

Soil suction profiles were taken in Test pits 1 and 2. Tests were carried out by the Tasmanian Department of Main Roads using a Wescor Pyschrometer. The dew point method was used for all samples. The results of the tests are shown on Figures F2 and F3.

TABLE F.1. ATTERBERG LIMITS AND CLAY FRACTION

Sample number	*Test pit or borehole (TP or BH)*	*Depth (m)*	*Atterberg Limits (%) liquid limit*	*plastic limit*	*plasticity index*	*Clay fraction (%)*
1	TP1	2.2 to 2.4	53	28	25	28
2	BH7	2.5 to 2.6	52	29	23	43
3	TP1	3.3 to 3.4	84	34	50	46
4	TP2	2.4 to 2.6	123	40	83	65
5	TP2	1.9 to 2.1	109	44	65	60
6 and 7	BH1	3.1 to 3.4	62	30	32	33 to 44
A	TP2	1.4 to 1.6	106	42	64	-
B	TP1	0.2 to 0.4	46	30	16	-
C	BH8	1.3 to 1.5	-	-	-	-
D	BHC	3.7 to 3.8	-	-	-	-

NOTES: Sample numbers refer to numbered curves on Figure F4.
Curve 6 and Curve 7 are analyses of the same sample.
Analysis 6 was carried out by the author and analysis 7 by the Hydro-Electric Commission, Tasmania.

TABLE F.2. BULK DENSITY AND DRY DENSITY

Sample number	*Test pit number*	*Depth (m)*	*Moisture content (%)*	*Dry density (t/m^3)*	*Bulk density (t/m^3)*	*Bulk unit weight (kN/m^3)*
1	1	2.3 to 2.3	30.9	1.41	1.84	18.1
2	1	2.8 to 2.9	33.4	1.45	1.93	18.9
3	1	3.3 to 3.4	37.1	1.33	1.83	18.0
4	2	2.3 to 2.4	43.2	1.15	1.64	16.1
5	2	2.7 to 2.8	37.5	1.35	1.86	18.2
6	1	0.2 to 0.3	28.0	1.53	1.96	19.2
7	1	0.2 to 0.3	28.5	1.50	1.93	18.9
8	1	0.3 to 0.4	30.5	1.53	1.99	19.5
9	1	0.3 to 0.4	29.0	1.53	1.98	19.4
10	1	0.3 to 0.4	30.9	1.52	2.00	19.6

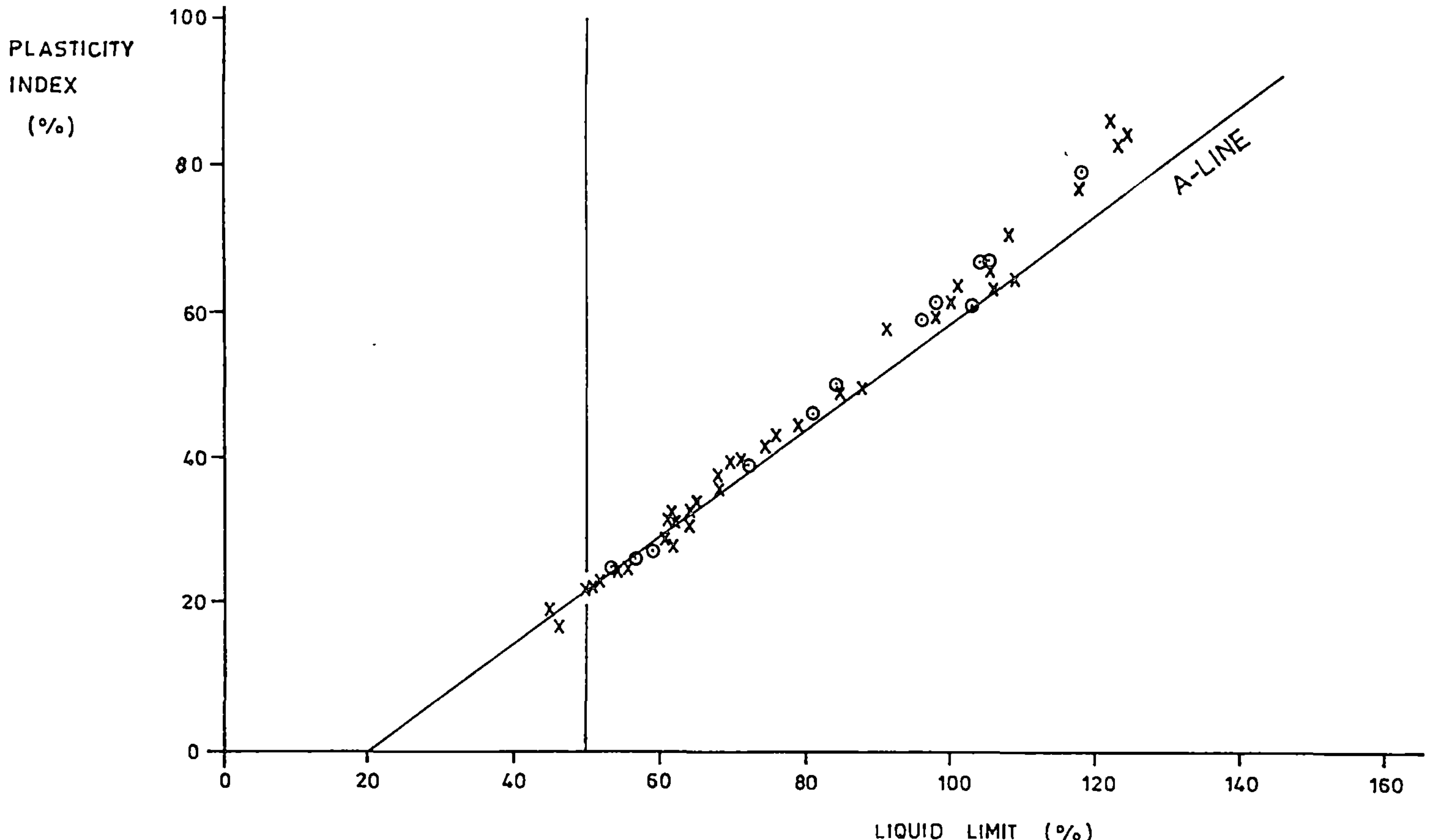
NOTES: Samples 1 to 5 were collected in summer
Samples 6 to 10 were collected in winter.

TABLE F.3. SUMMARY OF CONSOLIDATION TEST RESULTS

Load (kPa)	*Coefficient of consolidation, C_v (mm²/min)*		*Coefficient of volume change, M_v (m²/kN)*		*Coefficient of permeability, k (mm/sec)*	
	C1	C2	C1	C2	C1	C2
27.5	142	24.9	0.00021	0.00019	4.9×10^{-6}	7.7×10^{-7}
55	26.9	9.22	0.00026	0.00028	1.1×10^{-6}	4.2×10^{-7}
110	7.50	7.67	0.00025	0.00039	3.1×10^{-7}	4.9×10^{-7}
220	4.07	3.35	0.00017	0.00026	1.1×10^{-7}	1.4×10^{-7}
440	2.48	1.27	0.00013	0.00014	5.3×10^{-7}	2.9×10^{-8}
880	1.62	0.48	0.00007	0.0008	1.9×10^{-8}	6.3×10^{-9}
1760	2.18	0.29	0.00004	0.0005	1.4×10^{-8}	2.4×10^{-9}

NOTES: Sample C1 is from Test pit 1, 2.29 to 2.31 m. It has a plasticity index of 25%.

Sample C2 is from Test pit 2, 2.09 to 2.11 m. It has a plasticity index of 79%.



LEGEND

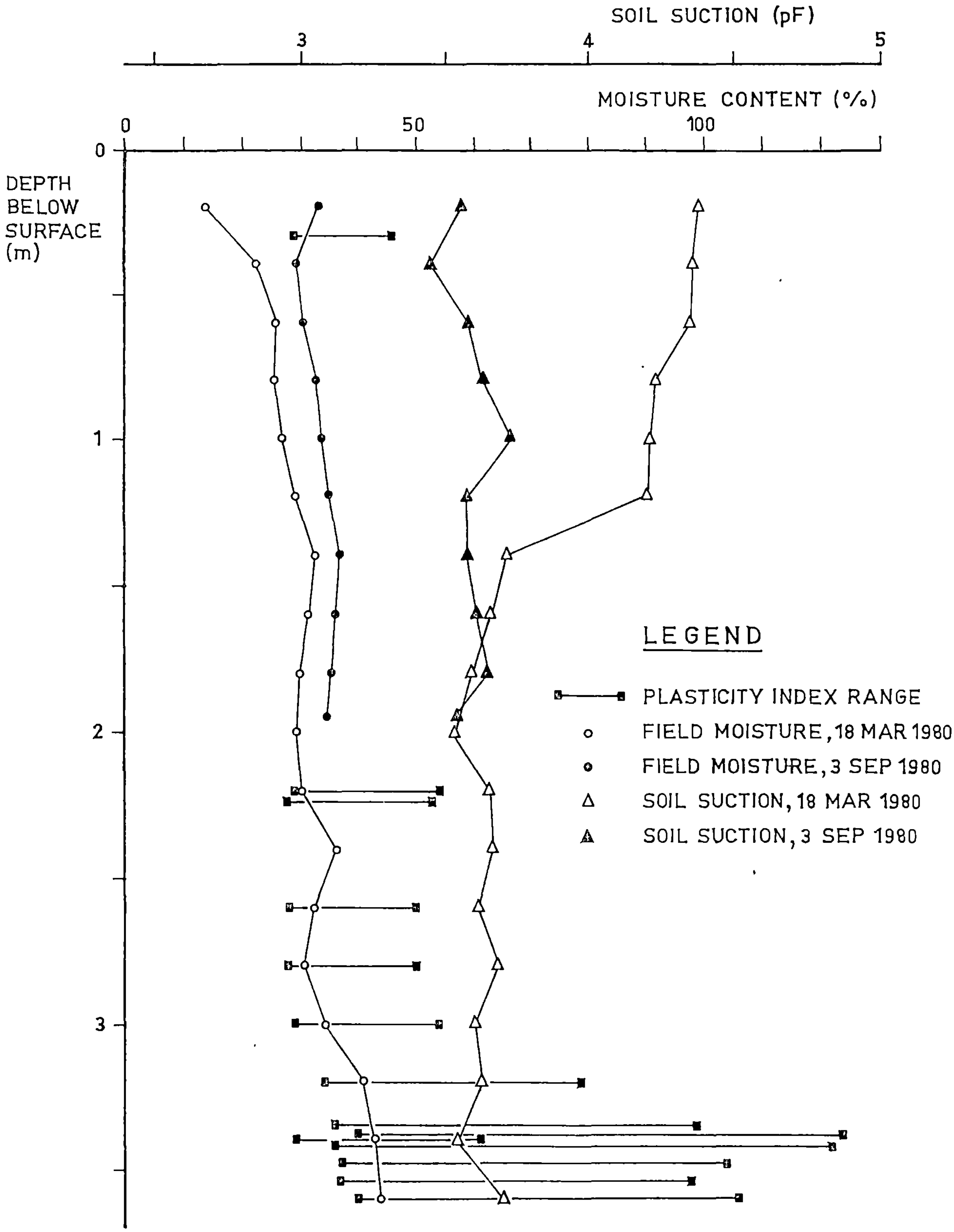
⊙ - SHEAR BOX SAMPLES

X - OTHER SAMPLES

BOVILLS SLIP

ATTERBERG LIMITS

FIG. F1

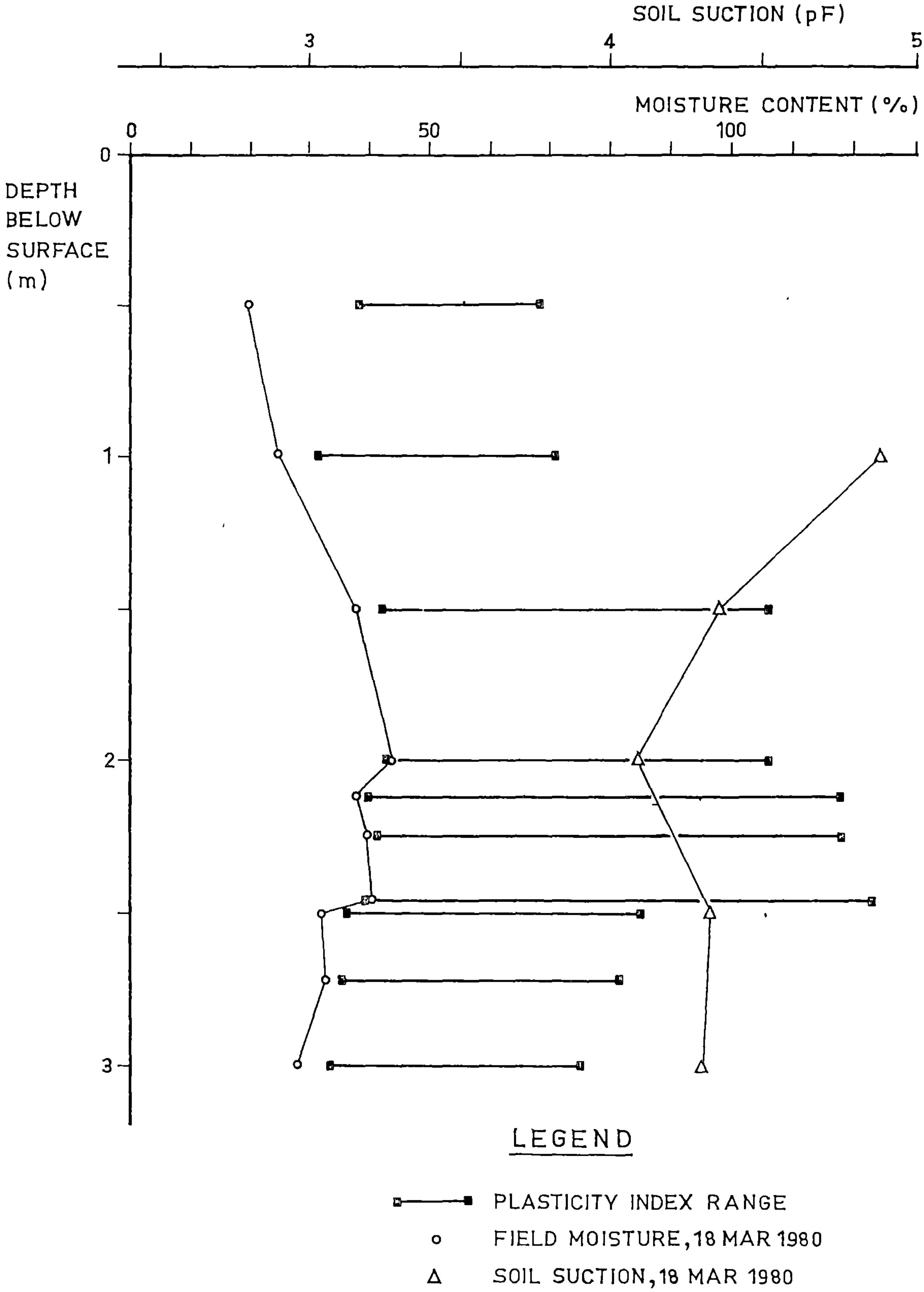


BOVILLS SLIP

TEST PIT 1 EXPLORATION

MOISTURE CONTENT ATTERBERG LIMITS & SOIL SUCTION PROFILE

FIG. F2

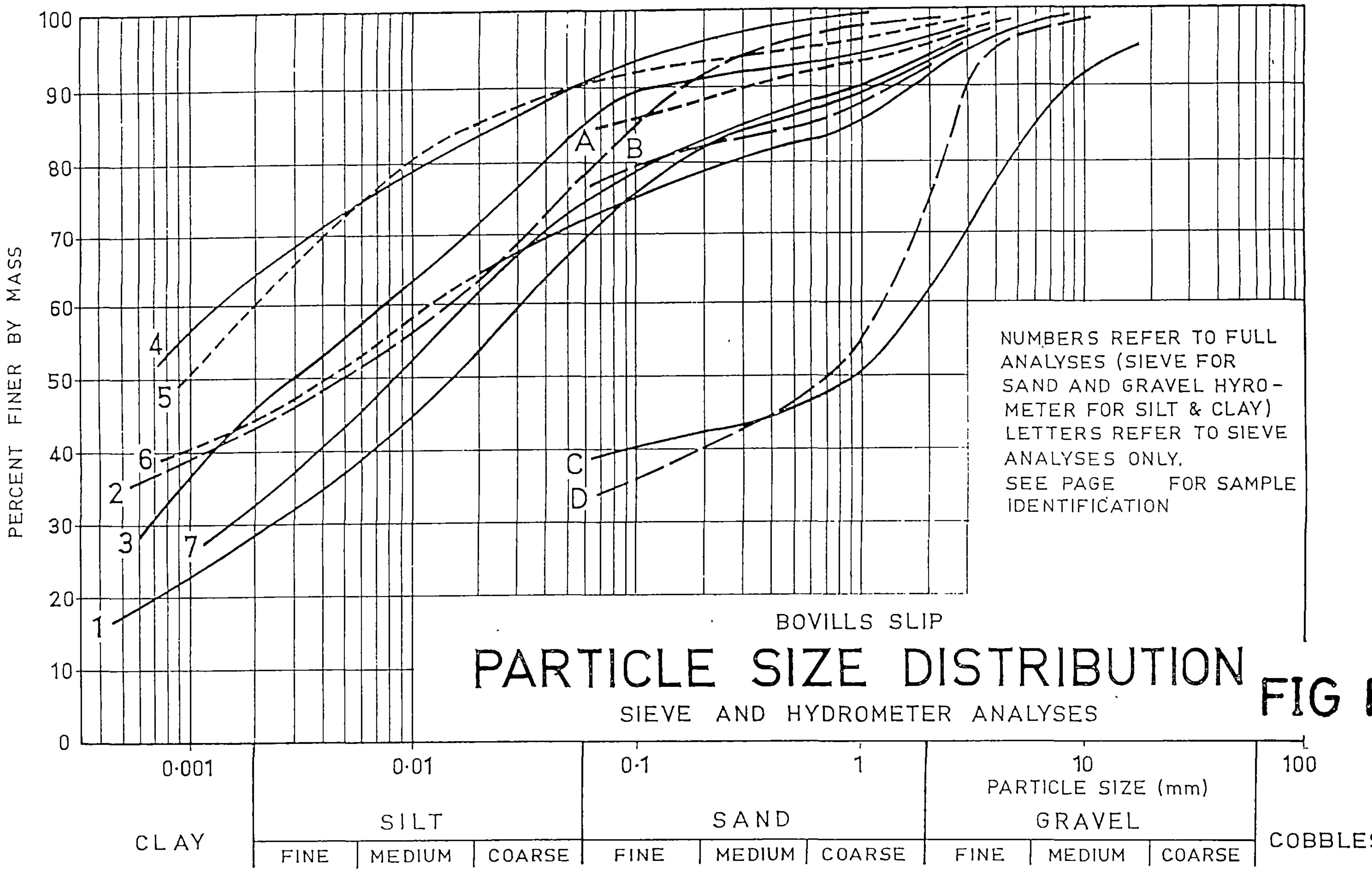


BOVILLS SLIP

TEST PIT 2 EXPLORATION

MOISTURE CONTENT ATTERBERG LIMITS & SOIL SUCTION PROFILE

FIG. F3

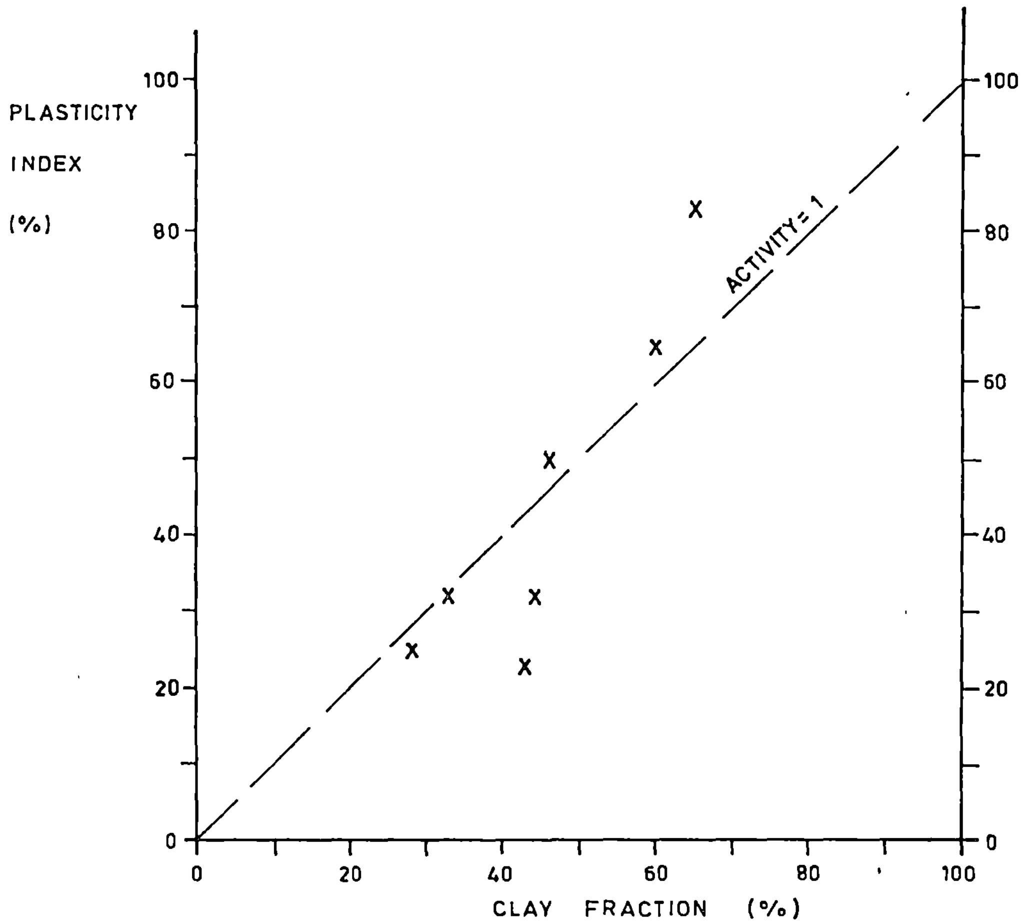


BOVILLS SLIP

PARTICLE SIZE DISTRIBUTION

SIEVE AND HYDROMETER ANALYSES

FIG F4

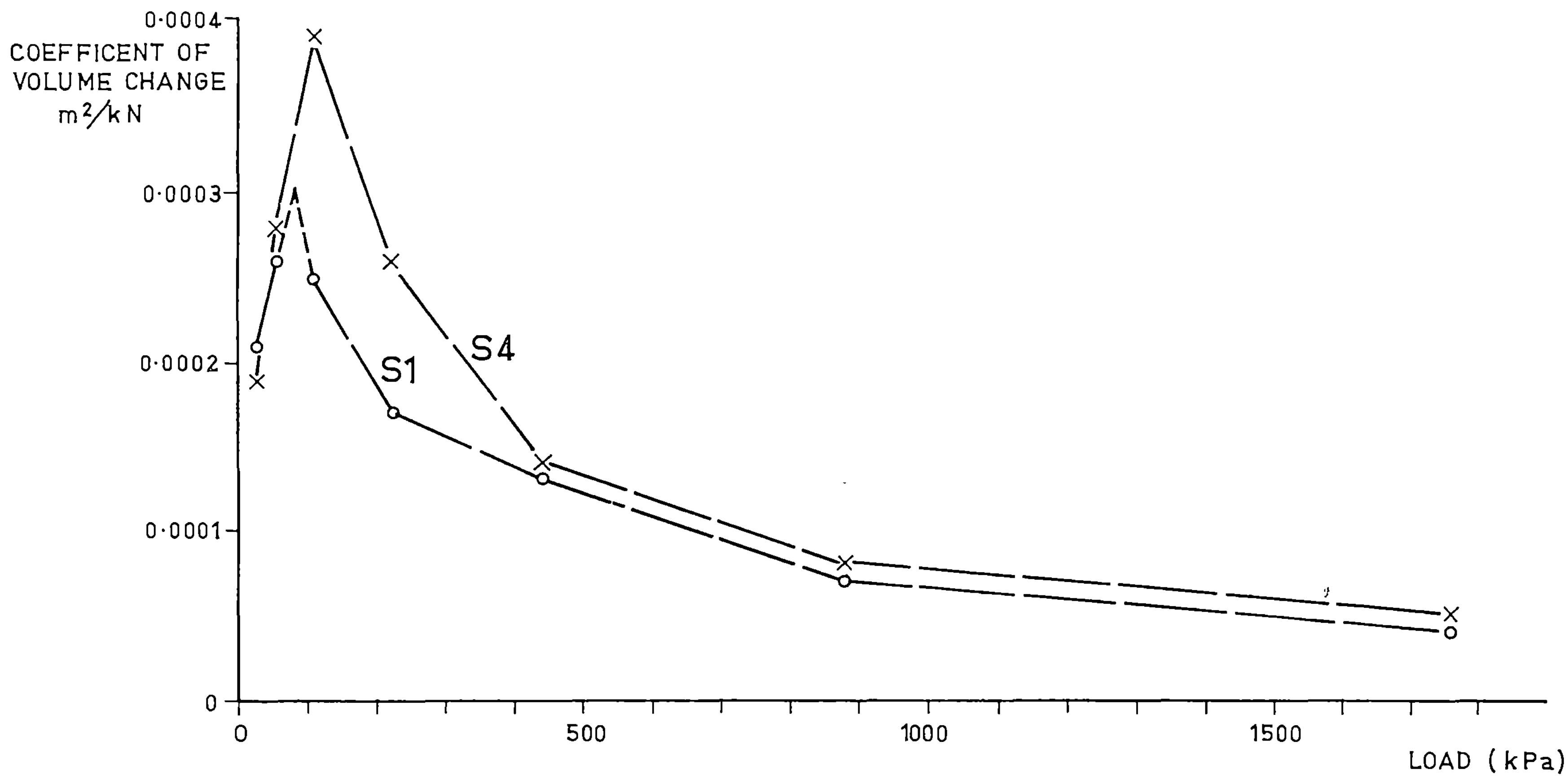


NOTE: CLAY FRACTION IS PERCENTAGE BY WEIGHT OF SOIL PARTICLES WITH A MEAN DIAMETER OF LESS THAN 2 MICRONS.

BOVILLS SLIP

RELATIONSHIP BETWEEN PLASTICITY & CLAY FRACTION

FIG. F5

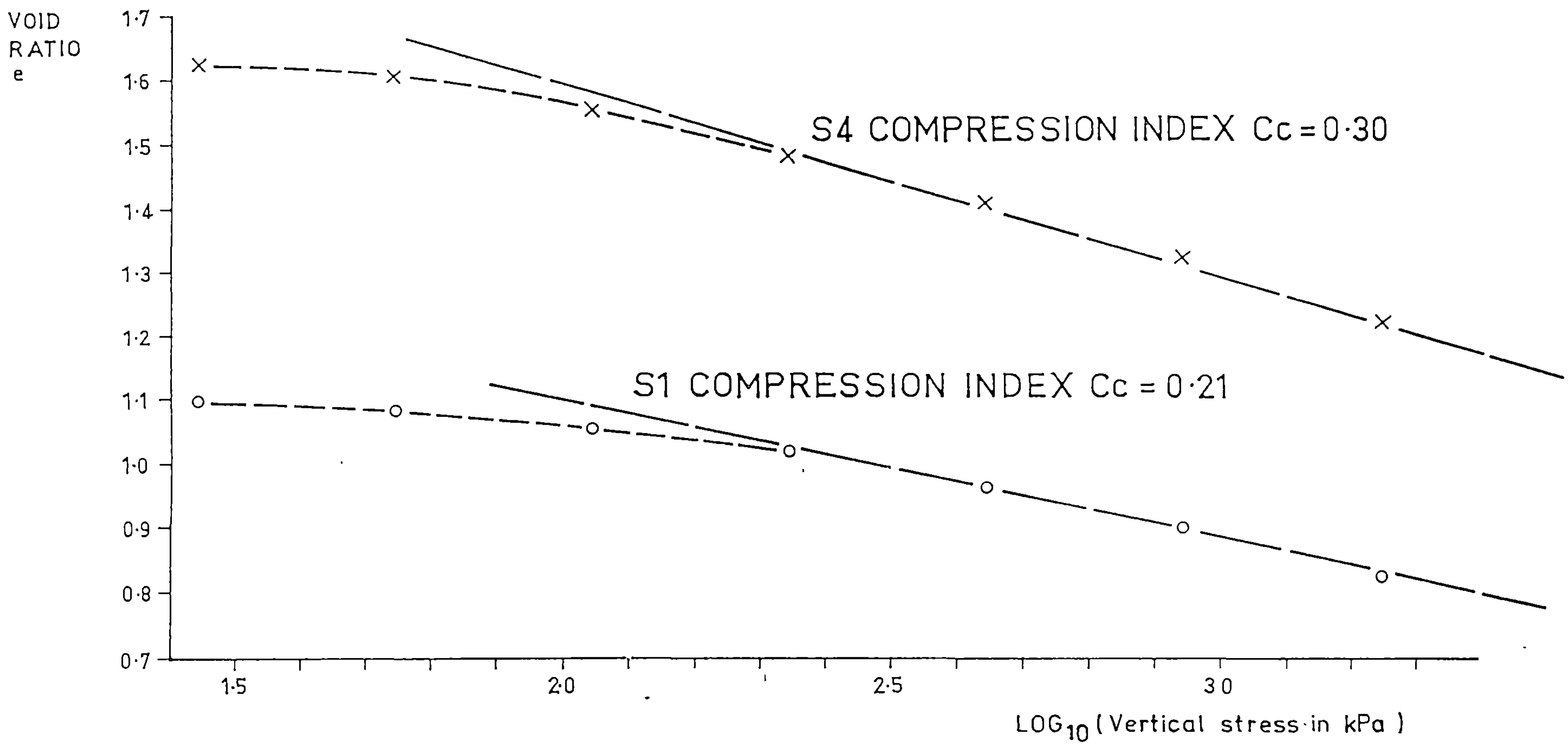


BOVILLS SLIP

CONSOLIDATION TESTS S1 & S4

COEFFICIENT OF VOLUME CHANGE v LOAD

FIG. F6



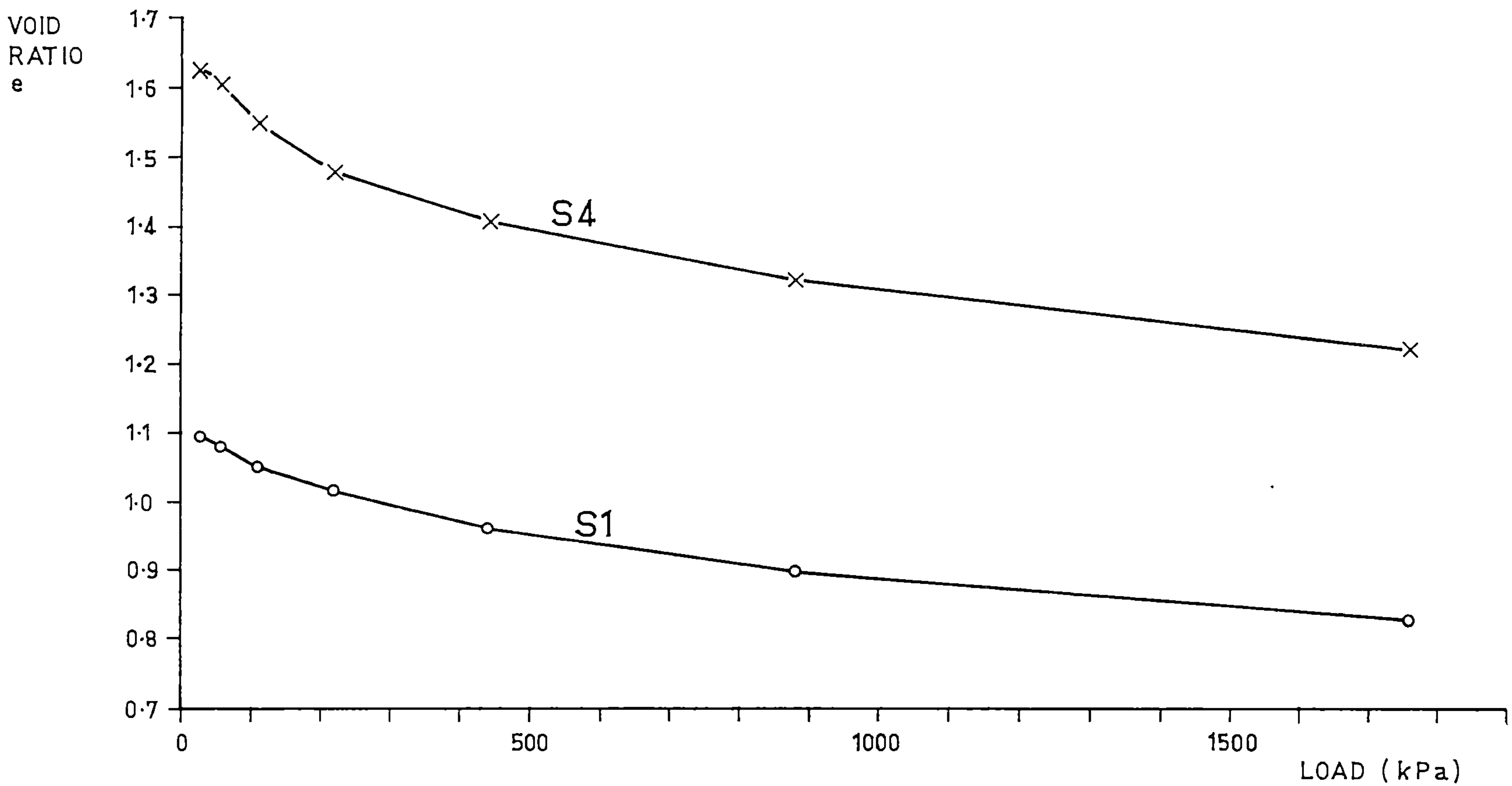
BOVILLS SLIP

CONSOLIDATION TESTS S1 & S4

VOID RATIO v LOG LOAD

FIG. F7

F14



BOVILLS SLIP

CONSOLIDATION TESTS S1 & S4

VOID RATIO v. LOAD

FIG. F8

APPENDIX G

MOVEMENT MONITORING

G.1 INTRODUCTION

In this Appendix, systems for monitoring surface and subsurface movements are described and some of the detailed results are presented. A summary of the recent movements affecting Bovills Slip is given in Chapter 6, Table 4. Figures are included at the end of this Appendix.

G.2 SURFACE MONITORING SYSTEMS

Two surface monitoring systems were used. The first consisted of a grid of five lines of wooden pegs, spaced at 5 m intervals. The position of the lines is shown in Figure 4. This grid was designed by the writer and established by G. Benn, a surveyor with the Department of Mines, Tasmania. Mr Benn has resurveyed the grid every 2 or 3 months since December 1979. Horizontal movements have been recorded relative to pegs on the flat parking area north of the road and vertical movements have been measured relative to the site datum on the foundations of a water storage tank about 200 m west of Bovills Slip. The grid has been tied into the Australian Metric Grid and levels have been tied into the Australian Height Datum.

In order to allow for easier and more frequent movement checks a second monitoring system consisting of four shorter lines was established. The lines are 12 m to 14 m long and consist of wooden pegs spaced less than 2 m apart. The position of the lines is shown in Figure 4 and cross profiles are shown in Figure G1. The lines were surveyed by measuring the distance between successive pairs of pegs with a metal tape glued to a 2 m aluminium rod. The slope angle between each pair of pegs was measured with the clinometer of a Brunton compass placed on the aluminium rod. The whole operation is simple and quick and can easily be carried out by one person. Over 50 repeat surveys have been carried out since February 1980 with most information being collected during the winter months. There are enough pegs to allow individuals that have been disturbed or lost to be replaced without losing control of the whole line.

G.3 SURFACE MONITORING RESULTS

A summary of the surface monitoring results is presented here. Full details of all the repeated surveys are available in files in the Department of Mines library.

Figure G2 shows the increase in total length of each of the four shorter lines (F, G, H, and J) plotted against time. Some of the steps are caused by individual pegs being removed or replaced and some represent slip movement. A detailed look at the results shows that minor movements or readjustments of the slip can occur in different parts of the slip at different times.

Seasonal changes in level of two of the survey pegs relative to the site datum are shown in Figure G3. Vertical movement is due to changes in moisture content of the top 1 m to 2 m of soil. Seasonal up and down movement is about 20 mm. This figure should be regarded as a minimum as the site datum itself may be subject to some movement.

The relative downslope movements of the three grid lines crossing the slip are shown in Figure G4. Most of the movement has been on the West Slip with some minor movement on the East Slip. Maximum total downslope movement since 1980 has been about 50 mm. There were movements of 10 to 20 mm during the winter of 1980 and movements of 20 to 30 mm in August 1981. There was no significant movement during the winter of 1982.

G.4 SUBSURFACE MONITORING

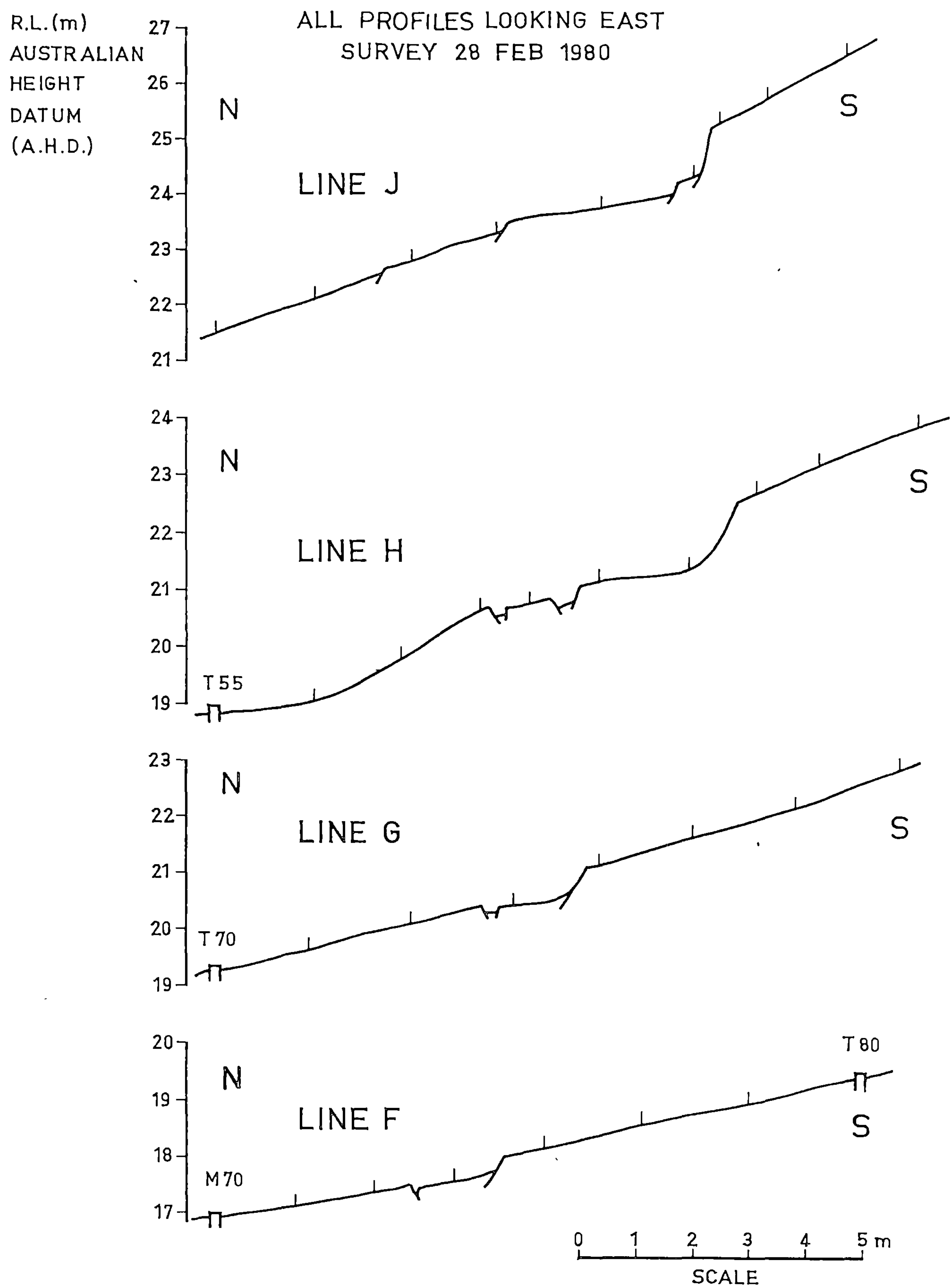
Subsurface movements have been monitored by regularly checking the PVC piezometer tubes for any deformation. A close fitting probe was able to pick up zones where the tubes deformed. Greater movement would cause rupture of the tubes and this could also be picked up with the probe. In August 1981 the slip moved about 25 mm at the surface and deformation of the piezometer tubes was detected in six of the boreholes (Table G.1). This allowed the base of the slip to be well defined.

TABLE G.1. AUGUST 1981 PIEZOMETER TUBE DEFORMATION

Piezometer	Depth of failure zone (m)
1	3.05
2	3.20
6	2.70
8a	3.65
B	1.42
C	1.22

G.5 CONCLUSIONS

Movement monitoring systems were successful in detecting surface and subsurface movements. If more information was required about the time and rate of movements surveys would have to be repeated more frequently or movement monitoring devices attached to continuous recorders could be used (Prior and Stephens, 1972). Subsurface movements could be measured more precisely with inclinometers (Mitchell and Eden, 1971).

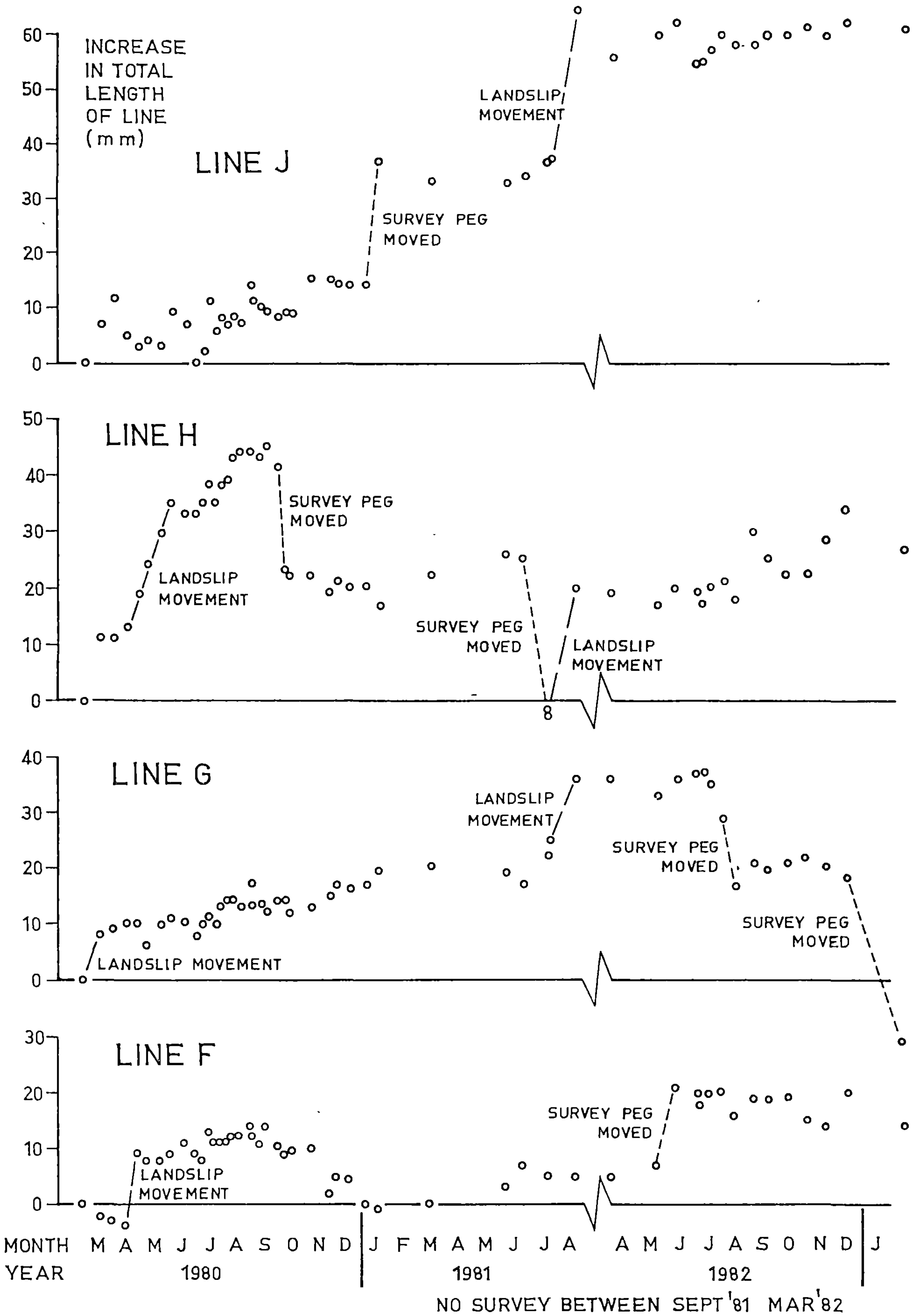


BOVILLS SLIP

MOVEMENT MONITORING

CROSS PROFILES OF LINES F, G, H, & J

FIG.G1

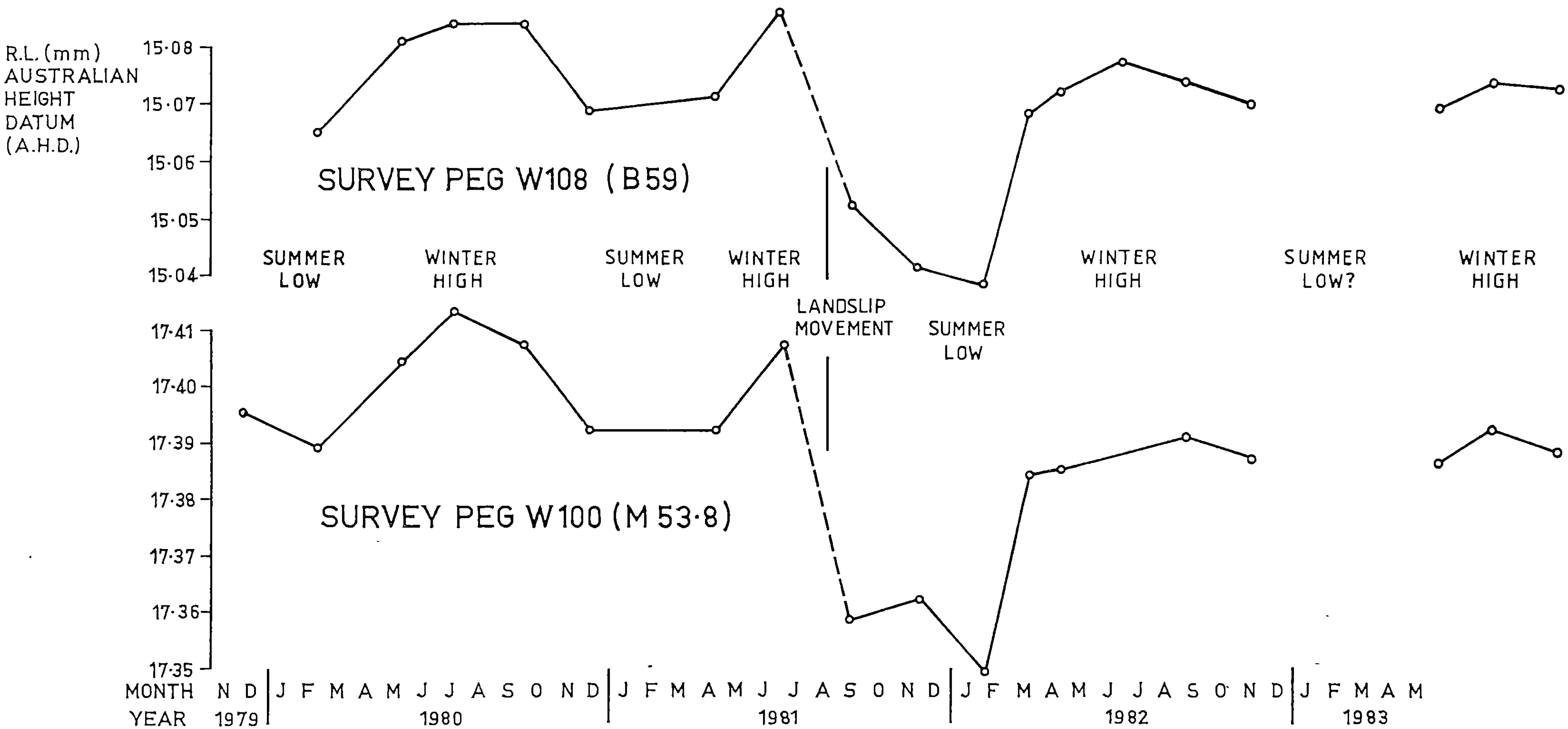


BOVILLS SLIP

MOVEMENT MONITORING

INCREASE IN LENGTHS OF LINES F, G, H, & J

FIG.G2

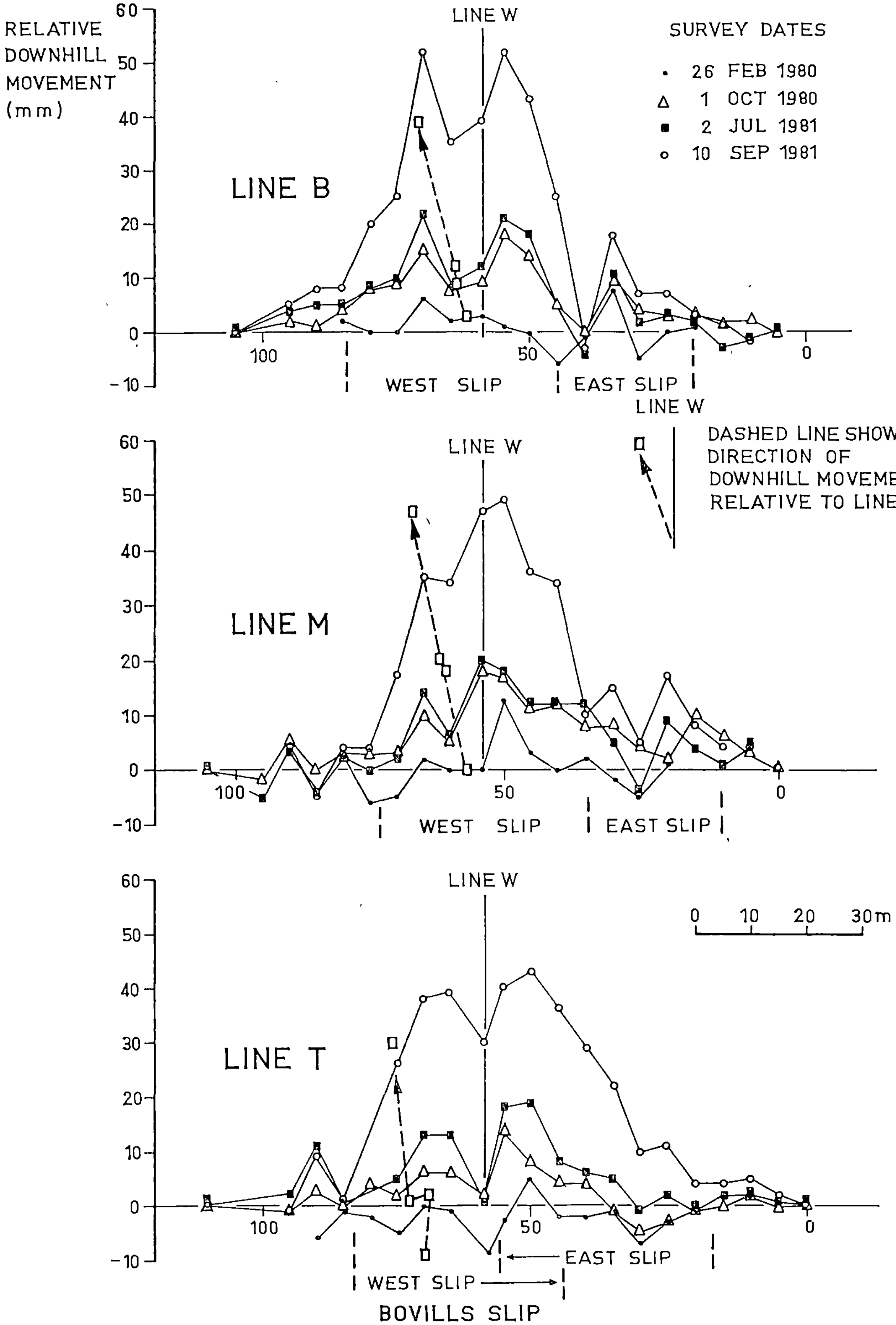


BOVILLS SLIP

MOVEMENT MONITORING

SEASONAL CHANGES IN R.L.

FIG. G3



MOVEMENT MONITORING

RELATIVE DOWNHILL MOVEMENTS LINES B,M & T

FIG.G4

APPENDIX H

ADDITIONAL PUBLICATIONS

Moon, A.T., 1983. Residual shearing mechanisms in natural soils. *Australian Geomechanics News,* Special edition for 5th ISRM Congress, 78-80.

Moon, A.T., in press. Effective shear strength parameters for stiff fissured clays. To be presented at the *Fourth ANZ Conference on Geomechanics,* Perth, 1984.

Residual Shearing Mechanisms in Natural Soils

A. MOON

Department of Mines, Tasmania

1. INTRODUCTION

A research project in progress at the University of Tasmania consists of a detailed field and laboratory investigation of a shallow landslip in cohesive soil. This paper discusses the results of the residual shear strength tests obtained during the investigation.

Lupini, Skinner and Vaughan (1981) demonstrate that the mechanism of residual shear changes with the nature and content of clay particles. These differences in mechanism result in significantly different values of residual shear strength.

The residual shear strength results from the present study are of interest because the three mechanisms identified by Lupini et al. were found in the one soil unit. For the particular soil studied there was a good correlation between plasticity index (which is directly related to clay content) and residual shear strength. A relationship between the fully softened strength and the residual strength was also apparent.

2. RESIDUAL SHEARING MECHANISMS

Lupini et al. demonstrate that the proportion of platy particles to rotund particles, and the coefficient of inter-particle friction of the platy particles, control the behaviour of a soil in residual shear. They describe three modes of residual shear as follows:

Turbulent Mode - in soils with a low proportion of platy particles . Preferred platy orientation does not occur.

Sliding Mode - in soils with a large proportion of platy particles. A low shear strength surface of strongly oriented low friction platy particles forms.

Transitional Mode - involves both turbulent and sliding shear.

Lupini et al. reached their conclusions after reviewing published correlations between residual friction angles and index properties and carrying out ring shear tests, electron micrographs, and thin section analyses on soil mixtures with artificially varied gradings.

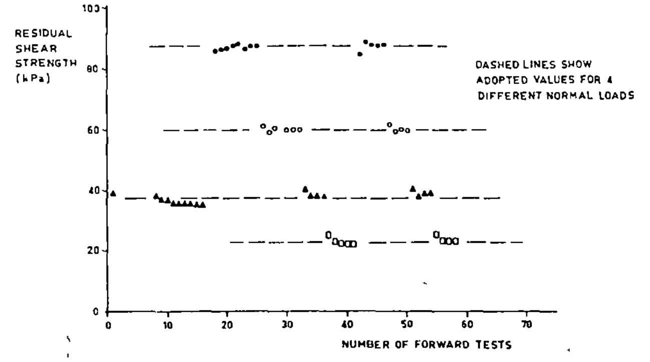


FIG. 1 RESIDUAL SHEAR STRENGTH RESULTS FROM ONE SAMPLE

3. PROJECT DESCRIPTION

The landslip investigated occurs at the base of a coastal scarp about 2km east of Devonport on the north coast of Tasmania. The coastal scarp has been cut into weathered olivine basalt of Tertiary age. The landslip occurs in weathered basalt colluvium which accumulated at the base of the slope during the Last Glaciation (Late Quaternary). The colluvium consists of high plasticity, red-brown silty clay with rock fragments. The landslip affects an area of about 3000m^2 and is up to 5m deep. Recent instability began after the construction of road works at the base of the slope in 1973 and slip movements have been recorded in most subsequent years.

The research project has involved field investigations of the geology, pore pressure, rainfall and slope movement. Laboratory investigations have included shear strength, grading, X-ray diffraction and index property tests.

4. RESIDUAL SHEAR STRENGTH

Residual shear strengths of samples in the silty clay colluvium were determined by testing undisturbed samples in a 60mm square reversing shear box. Multi-stage tests were performed using procedures similar to those described by Cullen and Donald (1971) and Chowdhury and Bertoldi (1977). Each sample was tested under four different loads consistent with overburden pressure. Tests were repeated at each load until a consistent value was obtained. Most of the tests were carried out with a box drive rate of 0.02mm/minute. A typical set of results for one sample is shown in Figure 1.

TABLE 1
SUMMARY OF RESIDUAL SHEAR STRENGTH TEST RESULTS

Group Number	Shearing Mechanism	Number of Tests	Residual Cohesion c'_R (kPa) mean	95% confidence limits	Residual Friction Angle φ'_R mean	95% confidence limits	R^2 (%)
1	Turbulent	5	3.6	1.1 to 6.1	28.3	27.1 to 29.4	100.0
2	Transitional	2	4.9	3.3 to 6.5	15.2	14.3 to 16.1	99.93
3	Sliding	8	3.7	1.3 to 6.0	10.0	8.6 to 11.3	99.94

NOTE: R^2 is a measure of the proportion of variation of the data that is explained by the assumption that the regression equation is linear.

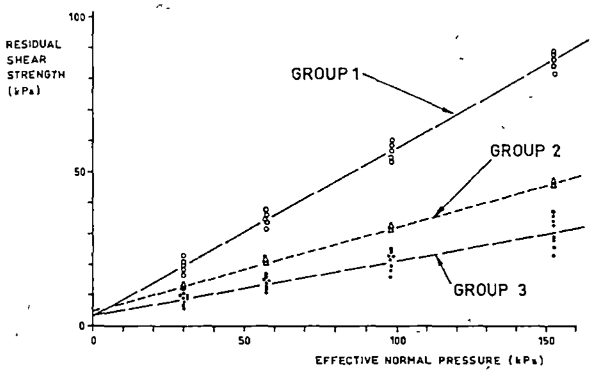


FIG.2 FIFTEEN RESIDUAL SHEAR STRENGTH TEST RESULTS

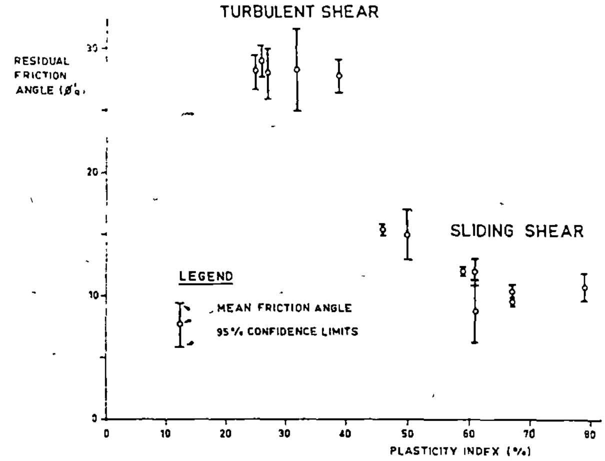


FIG 4. PLASTICITY AND RESIDUAL STRENGTH RESULTS

The results for 15 different samples are given in Figure 2 and Table 1. The friction angle results suggest that there are three quite different materials on the site. However, visual examination of the samples and other laboratory test results indicate that there is one soil type with a continuous variation of properties rather than three different soils. Atterburg limits results on the colluvial soil show a continuous variation over a wide range of plasticity (figure 3). Grading curves indicate that the clay fraction varies from about 30% to 65%. X-ray diffraction results show that montmorillonite and kaolinite are the main clay minerals in all of the samples.

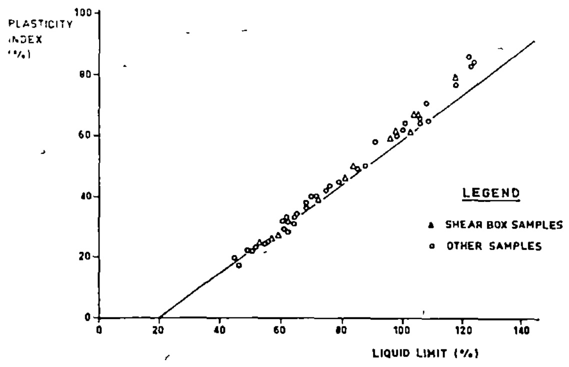


FIG. 3 ATTERBERG LIMITS RESULTS FOR SILTY CLAY COLLUVIUM

The relationship obtained between the residual shear strength and the plasticity index, as shown in Figure 4, is similar to that obtained by Lupini et al (1981). Up to a plasticity index of about 40% the samples failed by turbulent shear. Shear planes did not develop even after 60 or 70 reversals. Above a plasticity index of 55% the samples failed by sliding shear and developed polished and slickensided shear planes. The two intermediate results may be regarded as representing the transitional mode.

5. FULLY SOFTENED SHEAR STRENGTH

For the analysis of first time failures the most appropriate laboratory parameters are those for the "fully softened" or "critical state" condition (Skempton, 1977). In this project the fully softened strength parameters were determined by consolidated undrained triaxial tests with pore pressure measurements and by direct shear tests on undisturbed and normally consolidated remoulded samples. A comparison of the residual and fully softened shear strength parameters adopted for the project is given in Table 2.

TABLE 2
SHEAR STRENGTH PARAMETERS ADOPTED

	Shearing Mechanism			
	Turbulent Mode		Sliding Mode	
Parameter	c' kPa	φ' deg	c' kPa	φ' deg
Fully softened	3	30	3	21
Residual	3	28	3	10

NOTE: Shear strength parameters for transitional mode are intermediate between turbulent mode values and sliding mode values.

6. DISCUSSION

The recognition of the different shearing mechanisms has enabled the relationship between plasticity index and residual shear strength to be understood for one soil unit.

Table 2 shows that for soil which fails by turbulent shear, the difference between the fully softened parameters (appropriate for the first time failure) and the residual parameters (appropriate for repeated movements) is small. For soil which fails by sliding shear the difference is large.

If a slip occurs in soil which fails by turbulent shear, the residual shear strength is not likely to be much lower than the fully softened shear strength. Such a slip may stabilize through small changes in geometry or pore pressure. However, if the soil fails by sliding shear, there will be a large reduction in shear strength and instability may continue, unless remedial action is taken.

7. ACKNOWLEDGEMENTS

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Effective Shear Strength Parameters for Stiff Fissured Clays.

SUMMARY Shear box and triaxial tests have been used to investigate the effective shear strength of a stiff fissured clay of constant mineralogy but variable plasticity. Different residual shearing mechanisms were recognised in the shear box tests with significantly different values of residual strength. The fully softened strength parameters appropriate for the analysis of first-time slides were investigated by both triaxial and shear box tests. The lower plasticity samples had a higher strength than the higher plasticity samples. For the soil tested both the residual and fully softened effective friction angles showed a pattern of dependence on the plasticity. It may be possible to establish similar correlations for other soils if the results reflect different shearing mechanisms caused by grading variations within a soil of constant clay mineralogy.

1 INTRODUCTION

Stiff fissured clays commonly occur in the more populated areas of Northern Tasmania. In Launceston and the Tamar Valley the clays are lake sediments of Tertiary age. Along the north-west coast, a red-brown clay soil has developed on basalt of Tertiary age. Landslips are common in both areas.

The analysis of the long term stability of a natural slope, or the design of permanent cuttings in stiff fissured clays, require the knowledge of the appropriate effective shear strength parameters. These parameters have been investigated at a landslip in basalt soils near Devonport on the north-west coast of Tasmania. Multi-stage direct shear tests and consolidated undrained triaxial tests were used to determine the laboratory strength of undisturbed and remoulded samples of the soil.

Moon (1983) has reported the results of the investigation of residual strength by direct shear tests. He showed that the recognition of the different residual shearing mechanisms in the natural soil enabled a relationship between plasticity index and residual strength to be established. Residual shearing mechanisms are described in detail by Lupini, Skinner and Vaughan (1981) who worked with artificial soil mixtures.

In this paper the investigation of residual strength by direct shear tests is described in more detail. The definition of fully softened shear strength parameters which are appropriate for the analysis of first-time slides is considered and the relationship between laboratory determined parameters and those applicable to the field is discussed. The investigation of fully softened strength by both triaxial and direct shear tests is described. The paper presents the results of all of the strength tests and discusses the relationship between shear strength parameters and plasticity index for a soil of constant clay mineralogy but variable grading and plasticity.

2 DESCRIPTION OF SOIL

All of the samples tested were obtained from test pits and boreholes within the landslip. Field observations and laboratory tests indicate that the slip occurs within one soil unit of constant clay mineralogy. The soil has a continuous variation in plasticity due to variations in clay content. The soil consists of red-brown silty clay with minor rock fragments. Soil properties are summarised in Table I.

TABLE I

SOIL PROPERTIES

Liquid Limit:	46 to 124%
Plastic Limit:	28 to 44%
Plasticity Index:	17 to 84%
Clay Fraction:	30 to 65%
Activity:	0.53 to 1.28
Clay Mineralogy:	Montmorillonite and kaolinite

3 STRENGTH PARAMETERS REQUIRED

If a landslip already exists, or there are pre-existing shear surfaces, residual strength parameters are required (Skempton, 1964).

If there has been no previous failure the possibility of a 'first time' slide must be considered. Skempton (1970) suggested that the field strength of stiff fissured clay at first failure corresponded to the 'fully softened' condition which is reached when further deformation at constant stress fails to cause any further increase in water content. The fully softened condition may be taken as a practical approximation of the critical state. The peak strength of normally consolidated remoulded clay is also the theoretical limiting strength of a stiff fissured clay which has undergone complete softening.

In a review of the slope stability of cuttings in Brown London Clay, Skempton (1977) reports that the fully softened angle of friction is equivalent to the peak angle of friction determined by laboratory tests on undisturbed samples. However, values of cohesion determined in the laboratory generally overestimate fully softened cohesion (c'). Chandler and Skempton (1974) discuss the cohesion intercept obtained by back analysis, and argue that although the field cohesion at the time of first failure is small, it cannot be zero. They point out that the c' = 0 assumption leads to the conclusion that the limiting slope of a cut would be, contrary to practical experience, independent of depth. They suggest c' values of between 1 and 2 kPa for London Clay and Upper Lias Clay. These values are similar to the residual cohesion determined by laboratory tests.

In light of the above discussion the effective shear strength parameters appropriate for the analysis of first time slides are referred to in this paper as the fully softened parameters. The fully softened angle of friction (ϕ') is assumed to be equal to the peak angle of friction determined by laboratory tests while the fully softened cohesion (c') is assumed to be equal to the cohesion obtained in residual strength tests.

4 RESIDUAL SHEAR STRENGTH

4.1 Test Methods and Procedures

The results presented in this paper were obtained using a reversing shear box. It cannot be assumed that ring shear tests would give similar results.

Multi-stage tests were used as described by Cullen and Donald (1971) and Chowdhury and Bertoldi (1977). Shear strength was recorded during the forward travel of the shear box which was reversed by hand at the end of each run. Each sample was tested under four different normal pressures ranging from 30 to 150 kPa. Test procedures varied slightly but most samples were tested at least twice at each normal pressure. After each change of normal pressure the sample was left overnight to expand or consolidate before testing continued. The tests were carried out with a box drive rate of 0.02 mm min $^{-1}$.

4.2 Load Displacement Curves

The form of the load displacement curve depended on the mechanism of residual failure (Lupini, Skinner and Vaughan, 1981). Moon (1983) has shown that the samples with a plasticity index below 40% failed by turbulent shear and did not develop shear planes, while samples with a plasticity index above 55% failed by sliding shear and developed continuous shear surfaces. Samples which failed by turbulent shear had a higher residual strength and produced different load displacement curves to samples which failed by sliding shear. Typical load displacement curves for the two types of failure are shown in Figure 1. The peak values were only obtained on the first run for an undisturbed sample (Section 5.3.1.).

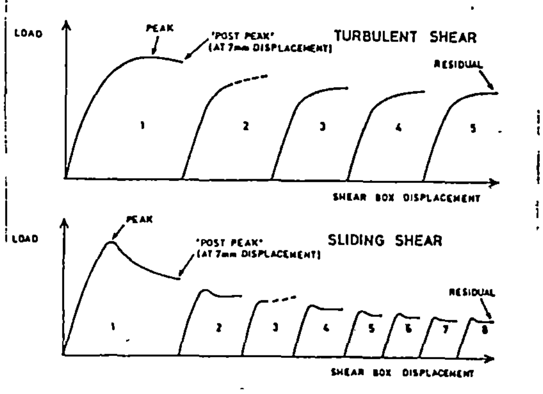

Figure 1 Typical load displacement curves

A number of forward runs were required to establish the residual strength at each normal pressure. There was a tendency for the load to drop a little from run to run until the residual state was reached. However, the load usually remained approximately constant (flat curve) during each run. After some experimentation it was decided to discontinue each run once the curve was flat and not to continue to an arbitrary displacement. This had the effect of increasing the number of runs that could be achieved each day and reducing the total testing time. In samples failing by sliding shear some of the later runs could be completed after less than 1 mm displacement.

4.3 Residual Shear Strength Results

Residual strength results for fifteen different samples are given in Table II. Values of effective residual cohesion (c'_r) and effective residual friction angle (ϕ'_r) were obtained by linear regression analyses. The assumption that the failure envelopes are linear in the range tested is justified by the high values of R^2. Residual cohesion varied but there was no significant difference between the values for the different shearing mechanisms.

TABLE II

RESIDUAL SHEAR STRENGTH RESULTS

Shearing mechanism	Plasticity index	Residual cohesion in kPa	Residual friction angle	R^2 %
Turbulent	25	0.5	28.2	99.96
(plasticity	32	3.5	28.3	99.79
index	27	3.2	28.1	99.92
<40)	39	7.3	27.8	99.96
	26	3.7	29.0	99.97
Transitional	46	5.1	15.4	99.99
	50	4.7	15.0	99.77
Sliding	59	4.2	12.0	99.99
(plasticity	61	3.1	12.0	99.91
index	-	2.6	7.7	99.88
>55)	67	2.8	9.6	99.99
	79	4.1	10.8	99.88
	-	3.1	8.4	99.68
	67	4.4	10.4	99.97
	61	4.9	8.8	99.11

R^2 is a measure of the proportion of variation of the data which is explained by the assumption that the regression equation is linear.

5 FULLY SOFTENED STRENGTH

5.1 Test Methods

Fully softened strength parameters were investigated by consolidated undrained triaxial tests and by direct shear tests. As discussed earlier (Section 3) laboratory strength testing on undisturbed samples may be expected to provide an estimate of the fully softened angle of friction (ϕ') but will generally overestimate the fully softened cohesion (c'). The five different methods used to determine ϕ' are shown in Table III.

Tests on undisturbed samples were preferred to tests on remoulded samples because remoulding destroys any diagenetic bonds or preferred particle orientation which may occur in natural soils.

TABLE III

METHODS USED TO DETERMINE FULLY SOFTENED STRENGTH

Apparatus	Sample Type	Failure Definition
Triaxial	Undisturbed	maximum ratio of principal stresses
Triaxial	Undisturbed	maximum difference of principal stress
Shear box	Undisturbed	peak strength
Shear box	Undisturbed	post peak strength (at 7 mm displacement)
Shear box	Remoulded	peak strength of normally consolidated sample

5.2 Triaxial Tests

5.2.1 Procedures

Consolidated undrained triaxial tests with pore pressure measurements were carried out and the results plotted on p-q stress path diagrams (Figure 2). The cell pressures were chosen to obtain strength parameters in the effective normal pressure range from about 20 to 200 kPa. A back pressure was applied to all of the samples and checks on the value of pore pressure parameter B indicated that the samples were fully saturated. The strain rate used was about 0.003% min^{-1}. The effects on the failure envelope of the restraint imposed by the filter paper drains and the rubber membrane were considered but appeared to have a negligible effect on ϕ'.

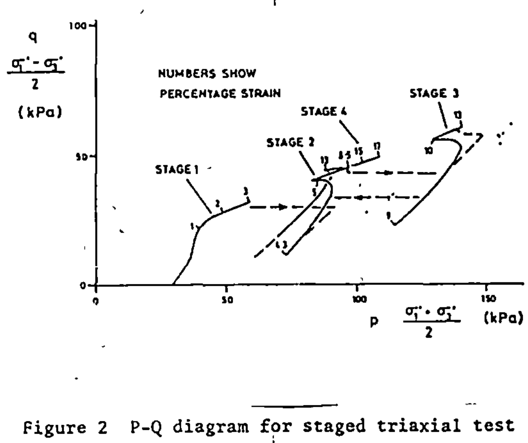

Figure 2 P-Q diagram for staged triaxial test

5.2.2 Definition of failure

Two definitions of failure were used. The first, the maximum ratio of principal stresses occurred at a low strain whereas the second, the maximum difference of principal stresses (deviator stress) occurred when the strain was significantly higher. The stress path between the two points follows the 'Coulomb line' and the sample may be regarded as being in a 'stabilised state of failure' (Kezdi, 1980). The different definitions of failure result in different values of c' and ϕ' (Table V).

5.2.3 Staged tests

Most of the consolidated undrained tests were staged with each sample being tested at four different cell pressures. Figure 2 shows that the stress path followed the Coulomb line over a large strain (1% to about 17%). In each test the cell pressure for the final stage was chosen to allow the stress path to cover the same range as in an earlier stage. In every case the Coulomb line from the final stage closely overlapped an earlier stage. Thus the Coulomb lines from each stage could be connected to form a single straight failure envelope.

5.3 Direct Shear Tests

5.3.1 Peak and post peak strength of undisturbed samples

For the first forward run of each shear box test the peak strength and the 'post peak' strength have been recorded (Figure 1). The post peak strength has been defined as the strength at the end of the first run which was standardised at a shear box displacement of 7 mm. The box drive rate used for these tests was about 0.005 mm min^{-1}. The post peak strength results are given in Table IV.

TABLE IV

POST PEAK STRENGTH RESULTS

Normal effective stress in kPa	Plasticity index %	Post peak strength in kPa	Plasticity index %	Post peak strength in kPa
30.0	25	22.7	60	21.2
30.0	27	21.7	61	18.3
57.2	27	36.0	59	31.1
57.2	33	31.9	67	26.0
57.2	39	39.7		
98.1	25	63.4	59	46.0
98.1	26	70.5	64	40.1
98.1	27	49.1	79	48.3
98.1	32	52.8		
152.6	25	92.5	59	68.0
152.6	26	101.5	79	63.1
152.6	27	86.4		

It was considered that the failure envelopes defined by the post peak strength would provide a better estimate of the fully softened friction angle. Many of the samples, which were collected in summer, may not have been fully saturated at the start of testing and scatter in the peak strength results could be due to variable increases in effective strength due to negative pore pressures. By the end of the first run (post peak strength), the soil in the failure zone would be likely to be closer to full saturation and negative pore pressures would be less. The results support this argument as the post peak strengths fit linear failure envelopes more closely than the peak strength results (R^2 in Table V).

5.3.2 Peak strengths of remoulded samples

A series of shear box tests was carried out on remoulded normally consolidated samples. Remoulded soil with a consistency close to the liquid limit was placed in the shear box and allowed to consolidate overnight before being tested. This process was repeated with consolidation and testing being carried out at four different normal pressures in

Effective Shear Strength Parameters for Stiff Fissured Clays.

TABLE V

RESULTS OF TESTS USED TO INVESTIGATE FULLY SOFTENED STRENGTH

Test Method	Plasticity index less than 40%				Plasticity index 50% or greater			
STAGED TRIAXIAL	cohesion in kPa	friction angle	R^2 %	number of samples	cohesion in kPa	friction angle	R^2 %	number of samples
maximum ratio of principal stresses	14.4	30.8	99.95		8.2	22.0	98.72 to 99.60	3
maximum difference of principal stresses	20.0	28.4	99.89		9.4	20.5	97.53 to 99.93	3
SHEAR BOX								
peak	6.5	30.6	99.26	12	15.7	22.9	95.06	9
post peak	2.8	30.4	99.76	12	7.8	20.7	99.91	9
remoulded					6.5	19.6	99.38	1

R^2 is a measure of the proportion of variation in the data which is explained by the assumption that the regression equation is linear.

the range from 30 to 150 kPa. The peak angle of friction has been taken as an estimate of ϕ' (Table V). The relatively low value of R^2 is caused by the slightly curved failure envelope which often results from tests on 'young' (i.e. remoulded) soils. This curvature of the failure envelope results in a lower estimate of ϕ' than that obtained from tests on undisturbed samples.

5.4 Fully Softened Shear Strength Results

The results of the investigation of fully softened strength parameters by triaxial and shear box testing are summarised in Table V. Soils with a plasticity index of less than 40% had a higher strength than soils with a plasticity index of 50% or greater. Thus the results were divided into two groups and analysed separately. The fact that the different methods of estimating ϕ' gave similar results increases confidence in the parameters obtained.

6 RELATIONSHIP BETWEEN SHEAR STRENGTH PARAMETERS AND PLASTICITY INDEX

The relationship between angle of friction (ϕ') and plasticity index (PI) for the soil tested is shown in Figure 3. The post peak results were obtained by analysing groups of samples with similar plasticity. Group A represents ϕ' obtained by linear regression analysis of test results obtained on eleven samples whose PI ranged from 25 to 33%. Group B represents the analysis of seven samples whose PI ranged from 59 to 67%. All the other results on Figure 3 represent single samples where multi-stage tests have resulted in the definition of separate failure envelopes for each sample.

The solid lines show the general pattern of results. The correlation between the residual angle of friction (ϕ'_r) and plasticity index has been explained by differences in residual shearing mechanism caused by variations in clay content (Moon, 1983).

The solid line indicating the relationship between the fully softened angle of friction (ϕ') and the plasticity index is less well established but can be justified on the following grounds. Up to a PI of 39% the test results indicate a ϕ' only slightly higher than ϕ'_r. Between a PI of 39% and 59% the only information is one remoulded test result which

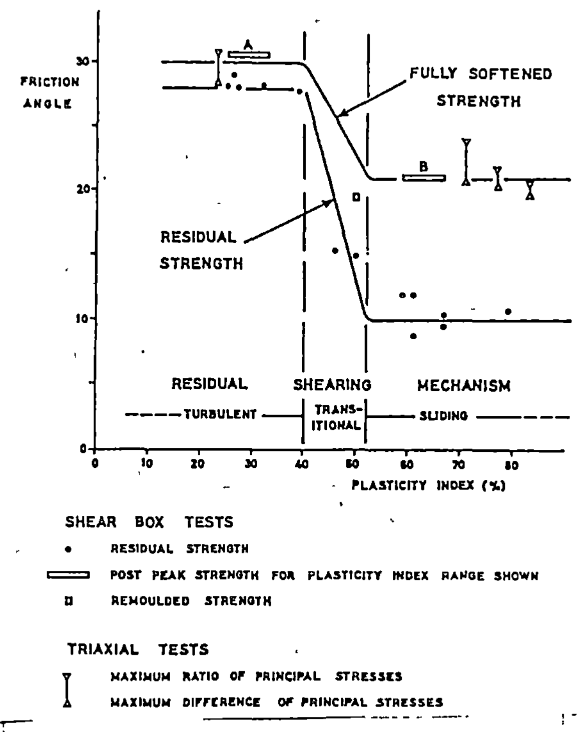

Figure 3 Relation between strength and plasticity

is likely to give a low estimate of $\bar{\phi}'$ because of the curved failure envelope (Section 5.3.2). For a PI of 59% and above the three triaxial tests could be interpreted as giving a sloping curve. However, the sample which gave the highest strength was tested at lower cell pressures than the other two samples and this may explain the slightly different results. The post peak shear box tests indicate a consistent strength over the range.

tested (Table IV). Lupini, Skinner and Vaughan (1981) tested sand-bentonite mixtures in a ring shear apparatus and found little variation in peak strength for clay fractions between 50 and 90%.

The cohesion (of about 3 kPa) obtained in the residual strength tests did not appear to be dependent on the residual shearing mechanism or the PI (Table II). The fully softened cohesion parameter is assumed to be similar to the residual cohesion (Section 3) and therefore also independent of the plasticity.

A summary of the relationship established between effective shear strength parameters and plasticity index is given in Table VI.

TABLE VI

SHEAR STRENGTH PARAMETERS AND PLASTICITY INDEX

	Plasticity index range (%)					
	Below 40		40 to 52		Above 52	
Parameter	c' kPa	ϕ' deg	c' kPa	ϕ' deg	c' kPa	ϕ' deg
Fully softened	3	30	3	21-30	3	21
Residual	3	28	3	10-28	3	10

The best estimate of the boundary between the middle and upper plasticity range is 52% (Table VI and Figure 3). The position of this boundary is not well defined and may lie anywhere between 50 and 60%.

7 CONCLUSIONS

It has been shown that the fully softened effective friction angle has a similar pattern of dependence on plasticity as previously demonstrated for the residual friction angle (Lupini, Skinner and Vaughan, 1981; Moon, 1983). Establishing the correlation between plasticity and strength depended primarily on the recognition of different residual shearing mechanisms. If the soil fails by turbulent shear, the fully softened strength will be slightly higher than the residual strength whereas if the soil fails by sliding shear the fully softened strength is likely to be much greater than the residual strength. For soils falling in the transitional zone both strength parameters will be sensitive to small changes in plasticity.

Effective strength testing is time consuming and expensive. The work of Lupini et al. (1981), Moon (1983) and the results presented here indicate how effective strength parameters may be determined with the minimum amount of such testing. Initial work should be aimed at establishing clay mineralogy, grading, and plasticity variations. Residual strength testing with shear box or ring shear apparatus should then be used to determine residual shearing mechanisms and residual shear strength parameters. Once the residual shearing mechanism is established the fully softened parameters may be investigated by either direct shear or triaxial testing.

Geological formations of stiff fissured clay, although varying in grading and plasticity, often have characteristic clay mineralogies. Using the approach suggested above it may be possible to determine a relationship between effective shear strength parameters and plasticity index which will be applicable for a whole region. Investigations of specific cuttings or slopes in such a region need only concentrate on recognising the appropriate shearing mechanism.

8 ACKNOWLEDGEMENTS

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EFFECTIVE SHEAR STRENGTH PARAMETERS FOR STIFF FISSURED CLAYS

KEYWORDS: Cohesion; consolidated undrained tests; direct shear tests; friction angle; fully softened strength; residual strength; shear strength; stiff clays; test procedures; triaxial tests.

ABSTRACT: Shear box and triaxial tests have been used to investigate the effective shear strength of a stiff clay of constant mineralogy but variable plasticity. Different residual shearing mechanisms were recognised in the shear box tests with significantly different values of residual strength. The fully softened strength parameters appropriate for the analysis of first-time slides were investigated by both triaxial and shear box tests. The lower plasticity samples had a higher strength than the higher plasticity samples. For the soil tested both the residual and fully softened effective friction angles showed a pattern of dependence on the plasticity. It may be possible to establish similar correlations for other soils if the results reflect different shearing mechanisms caused by grading variations within a soil of constant clay mineralogy.

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APPENDIX I

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